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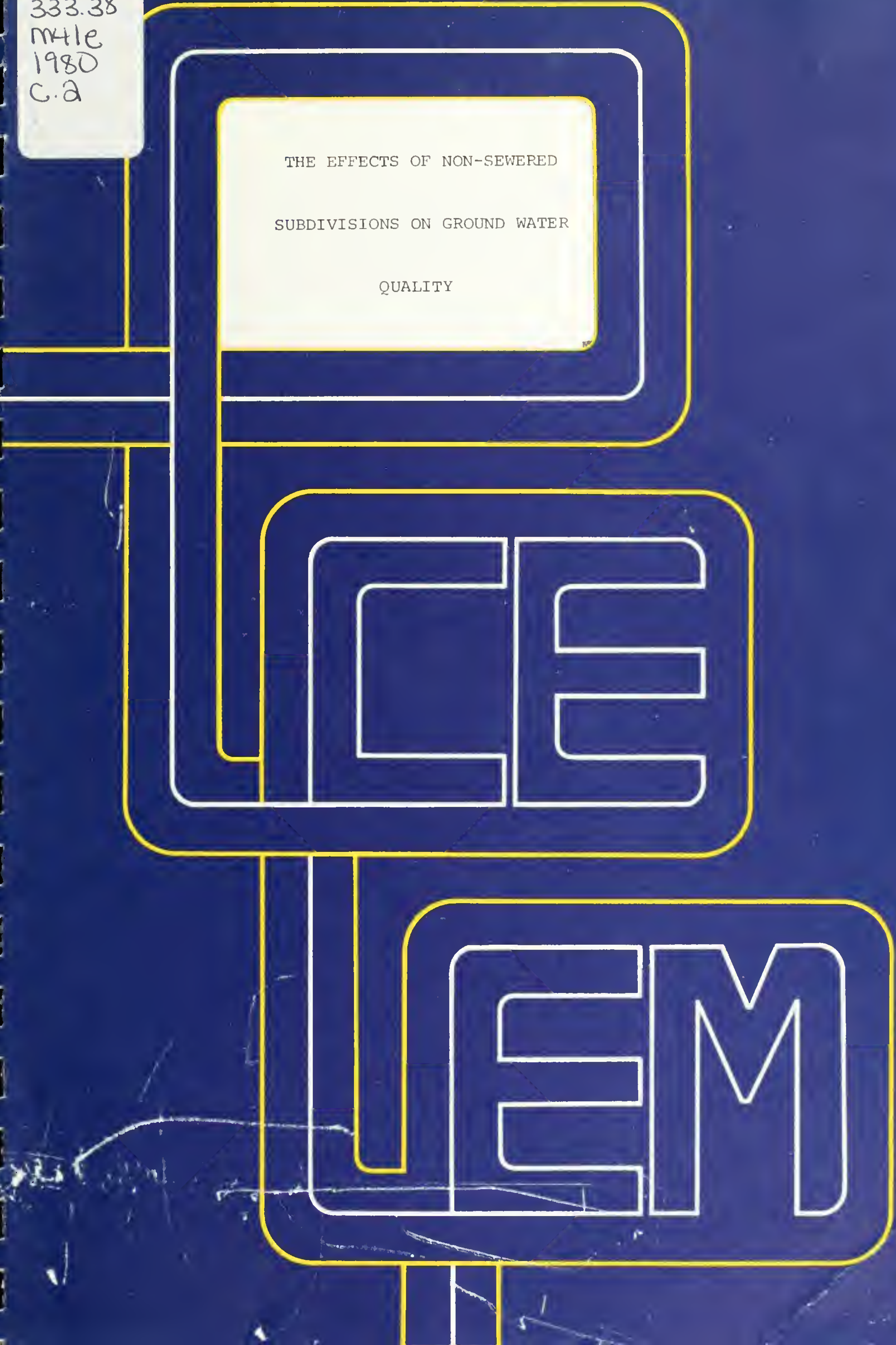
THE EFFECTS OF NON-SEWERED SUBDIVISIONS ON GROUND WATER QUALITY



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THE EFFECTS OF NON-SEWERED SUBDIVISIONS ON GROUND WATER QUALITY

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by

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for the

Water Quality Bureau
State Department of Health
&
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TABLE OF CONTENTS

INTRODUCTION 1

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS 3

SEPTIC TANK-SOIL ADSORPTION SYSTEMS: A LITERATURE SURVEY 5
- Purification Processes 5
  - Bacteria 5
  - Virus 8
  - Nitrogen 11
  - Phosphorus 14
  - Biodegradable Material 16
  - Other Constituents 17
- Failure of Soil Adsorption System 18
- Effects of Failure on Groundwater Quality 20

THE MONTANA PROJECT 23
  - Site Description 23
  - Installation of Sampling Points 25
  - Sampling Techniques 26
- Sampling Frequency 27
  - Analysis 27
  - pH 28
  - Specific Conductance 28
  - Chloride 30
  - Nitrate and Ammonia 31
  - Chemical Oxygen Demand 31
  - Phosphate 32
  - Fecal Coliform 32
- Data Interpretation 33

EVALUATION OF MONTANA CODES 46

ALTERNATIVE SOLUTIONS 49
- Alternative Individual Sewage Disposal Systems 49
- Systems with Possible Application in Montana 50
  - Mound Systems 50
  - Evapotranspiration Systems 54
  - Cluster Development with ST-SAS 56
- Systems Not Recommended for Use in Montana 57
  - Sand Filters 57
  - Aerobic Aeration Units 59
  - Recycling Systems 61
- Flow Reduction Techniques 62
  - Non-Functional Water Use 62
  - Water-Saving Devices 62

LIST OF TABLES

Page

1. Typical Septic Tank Effluent Characteristics 6
2. Sampling Frequency 27
3. Average Parameter Values 29
4. Summary of Design Criteria for Soil Adsorption Systems in Montana 46
5. Soil and Site Factors that Restrict Mound Construction
6. Water Saving Fixtures and Appliances 64

LIST OF FIGURES

Page

1. Entropy Acres
Individual Well Point Averages 34

2. Entropy Acres
Individual Residential Well Averages 34

3. Hawker Lane
Individual Well Point Averages 35

4. Hawker Lane
Individual Residential Well Averages 35

5. Lambkins
Individual Well Point Averages 36

6. Lambkins
Individual Residential Well Point Averages 36

7. Livingston
Individual Well Point Averages 37

8. Livingston
Individual Residential Well Averages 37

9. Sewell
Individual Well Point Averages 38

10. Sewell
Individual Residential Well Averages 38

11. Entropy Acres
Individual Well Point Averages 39

12. Entropy Acres
Individual Residential Well AVerages 39

13. Hawker Lane
All Sites Averaged Vs. Time, Well Points 40

14. Hawker Lane
All Sites Averaged Vs. Time, Residential Wells 40

15. Lambkins
All Sites Averaged Vs. Time, Well Points 41

16. Lambkins
All Sites Averaged Vs. Time, Residential Wells 41

17. Livingston
All Sites Averaged Vs. Time, Well Points 42

Page

18. Livingston
All Sites Averaged Vs. Time, Residential Wells 42

19. Sewell
All Sites Averaged Vs. Time, Well Points 43

20. Sewell
All Sites Averaged Vs. Time, Residential Wells 43

INTRODUCTION

It has been estimated that 95% of the United States' freshwater reserves are contained in groundwater aquifers (Sikora, et. al., 1976) and that at least one half of the U.S. population depends on groundwater as a source of drinking water. Statistics indicate that a sizeable portion of the nation's total groundwater supply contains increasing amounts of soil-treated wastewater. Keeley (1977) estimated that more than 17 million families in the U.S. (30% of the U.S. population) are served by septic systems or other means of subsurface sewage disposal. These families daily introduce a total of nearly 3.5 billion gallons of wastewater into the subsurface soil environment. This is the largest source of artificial groundwater recharge in the United States and, considering the nature of the percolated water, represents a potentially significant non-point source of pollution to underlying aquifers.

This nationwide potential problem also exists at the local level. Approximately 37% of Montana families currently use subsurface waste water disposal facilities (van der Leeden, 1975). The proliferation of subdivisions in Montana, many of which propose to employ individual septic tank-soil adsorption systems increases the concern for ground water quality.

There have been many studies which investigated the interaction of septic tank effluent and soils of various types. Several studies have been reported which relate groundwater quality to drainage from one chosen soil adsorption system. A few studies have attempted to relate ground water quality to widespread use of soil adsorption systems over a larger area. These studies all provide useful information in assessing the extent to which subsurface disposal effects ground water quality. However, unique conditions in Montana may result in unique problems. For instance, many

subdivisions in Western Montana are located in valleys on alluvial and/or glacial till soils. Surrounding mountains and foothills produce groundwater tables with steep gradients which are often near the surface. Thus, results of studies conducted in other parts of the U.S. may not correlate well with conditions in Montana.

In order to assess adequately the effects of unsewered subdivisions on ground water quality in Montana, the Montana State Department of Health and Environmental Services contracted with the Department of Civil Engineering and Engineering Mechanics at Montana State University to conduct an investigation of groundwater quality beneath five subdivisions in Western Montana. This report is the culmination of that investigation.

The study was conducted by University personnel between July 1978 and October 1979. This report includes a summary of the available literature concerning the interaction of soil and septic tank effluent and reports the results of other studies regarding groundwater quality and subsurface disposal of wastewater. Other sections describe the Montana study, give the results and conclusions of the study, evaluate the effectiveness of Montana's subdivision codes, and discusses alternatives to standard soil adsorption systems in problem areas.

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

Five subdivisions were selected for study by the Montana State Department of Health and Environmental Sciences. The subdivisions sizes ranged from approximately 27 acres to a few hundred acres. Population densities varied from approximately two to more than five persons per acre. Seasonal ground water ranged from 11 feet to less than two feet below the surface, with the higher groundwater levels being predominate.

Goundwater sampling stations were established by driving well points into the top portion of the groundwater table. Additionally, wells in use for domestic water supply were sampled along with the well points. Arrangement of sampling stations were co-axial with the groundwater flow from the upstream to the downstream boundaries of the subdivisions.

A total of 299 samples were taken from 41 sites and analyzed for eight parameters each. Results of the analyses indicate no significant water quality degradation as a result of the individual sewage disposal systems employed in the subdivisions. The writers of this report conclude that existing regulations governing the installation of soil adsorption systems for wastewater disposal from individual residences are adequate for most applications in Montana.

Restriction of subdivision development due to groundwater quality considerations may conflict with optimum land use planning. Alternative wastewater disposal systems for individual homes or clusters of individual homes are available which will allow residential development of substandard and marginal areas and still provide water quality and public health protection. It is recommended that regulations be prepared for these alternative systems and that their installations be encouraged in appropriate circumstances. In addition, since reduction of wastewater flow will lessen the possibility of groundwater contamination by either ST-SAS (septic tank - soil absorption systems) or alternative ISDS (individual sewage disposal system), it is recommended that water conservation practices be actively encouraged in homes not served by collective sewage treatment systems.

Continued monitoring of the established sampling locations would be beneficial. Long term trends could not be established in the short time span covered by the sampling program. Additionally, the subdivisions studied were experiencing a growth in their respective populations, thereby increasing the possibility of ground water contamination in the future. A twice yearly sampling schedule would develop long term trends.

SEPTIC TANK-SOIL ADSORPTION SYSTEMS

A LITERATURE SURVEY

The most widely accepted individual sewage treatment system (ISDS) in the U.S. consists of a septic tank followed by a soil adsorption field (usually a perforated-line drainfield).

The septic tank's main function is solids removal in which settleable organic materials are removed and slowly digested under anaerobic conditions. A septic tank usually removes up to 60% of the Biochemical Oxygen Demand (BOD), and up to 70% of the suspended solids (Beatty et al, 1973) in domestic wastewater. The effluent from septic tanks may be of varying quality, depending upon the character of the influent waste water. Viraraghaven, et. al. (1976) reported typical effluent characteristics as shown in Table 1 on the following page.

The effluent from the septic tank is transferred to the system's soil adsorption field, the most crucial part of the ISTS. It is here that the physical, chemical, and biological reactions occur which further purify the wastewater. It is also from the soil adsorption system that percolation to the groundwater occurs. Thus, the degree of purification which occurs in the soil mantle is extremely important.

PURIFICATION PROCESSES

Treatment processes in the soil mantle are varied and complex, with different processes influencing different parameters. The following discussion summarizes the treatment mechanisms for the most important parameters in septic tank effluent.

Bacteria

Bacterial travel through the soil-water environment is a function of soil texture and degree of saturation. Effectiveness of removal of bacteria

TABLE 1. TYPICAL SEPTIC TANK EFFLUENT CHARACTERISTICS

Characteristics	Range	Mean	Standard Deviation
pH	6.53-7.45		80
TSS, mg l^{-1}	68-624	176	85
BOD, mg l^{-1}	140-666	280	245
COD, mg l^{-1}	240-2026	568	245
SOC, mg l^{-1}	24-190	73	35
Total phosphates (PO_4-P), mg l^{-1}	6.25-30.0	11.6	3.5
Ammonia nitrogen, mg l^{-1}	76-111	97	12
Nitrate-N, mg l^{-1}	0.00-0.10	0.026	0.02
Total soluble Fe, mg l^{-1}	0.000-20.0	2.63	2.2
Chlorides, mg l^{-1}	37-101	53	15
Total coliforms $(100\ ml)^{-1}$	$0.24x10^{6}-24x10^{6}$	$2.3x10^{6}$[a]	
Fecal coliforms $(100\ ml)^{-1}$	$4100-5.2x10^{6}$	$1.6x10^{5}$[a]	
Fecal streptococci $(100\ ml)^{-1}$	$14000-7.4x10^{5}$	$1.1x10^{5}$[a]	
Pseudomonas aeruginosa $(100\ ml)^{-1}$	<0.2-16000	30[a]	

[a] Geometric mean values.

From Vararaghavan, T. and R.G. Warnock "Ground water pollution from a septic tile Field," Water, Air and Soil Pollution, vol 5 (1976b), p. 284.

in the soil is dependent upon physical filtration mechanisms (such as, straining, sedimentation, and adsorption) and organism survival rates (Gerba et al., 1976; Romero, 1970). Though limited data is available on organism survival in groundwater, the primary influencing factors appear to be soil moisture content, temperature, pH, nutrient availability, and predation by existing soil bacteria (Gerba et al., 1976; Romero, 1970). Bouma (1972) reported that the biological clogging mat ("crust") developed in the treatment field under aerobic conditions is very efficient in adsorbing and holding bacteria. Low loading rates also significantly enhance removal of fecal indicators (Ziebell et al., 1975). While Gerba, et al. (1976) reported that survival of enteric bacteria under adverse conditions seldom exceeded 10 days, Romero (1970) and Gerba (1976) both reported that survival under favorable field conditions may extend up to approximately 100 days.

Since many soils having pore openings several times larger than the size of bacteria (0.001 millimeters) have very efficient bacteria removal, straining is obviously not the only method of removal of bacteria in the soil. Removal in these soils appears to result from adsorption onto the surface of soil particles. This adsorption takes place in spite of the fact that bacteria are hydrophilic (water-loving, usually with a layer of bound water at the surface) colloids which possess a net negative charge at the surface (Goldsmith et al. 1972). Since most soils also carry a net negative charge, one would expect repulsion rather than attraction of the two particles. Factors associated with other sewage components appear to interfere with these electrical forces, however. Goldsmith et al. (1972) reported that the surface charge results from the dissociation of carboxylic and amino groups at the surface of the bacterium. Thus, in water with a high ionic strength and neutral or slightly acidic pH, such as that found in a septic tank effluent, cations (Ca^{++}, Mg^{++}, Na^{+}, H^{+}) in the water neutralize and sometimes supersaturate the surface of the bacteria, making them susceptible to adsorption by negatively charged soil particles. Experiments using desert sand have shown that even coarse sand can be effective in removing bacteria from water if the water is in the acid pH range and has sufficient ionic strength (Goldsmith et al. 1972).

Observed bacterial travel through soils varies greatly due to different soil-water characteristics. Bacteria travel only a few feet in effluent percolating under unsaturated soil conditions, but up to several hundred feet in saturated soil (Gerba et al., 1976; Foster et al., 1973; Bouma, 1972). Reports of travel ranging from 1500 feet to 2000 feet in coarse gravels and fractured bedrock are also common (Romero, 1970; Merrell et al., 1967; and Allen et al., 1973). Magdoff et al., (1974) observed removal of all fecal indicators (applied at over 3.8×10^4 organisms per 100 mls) in a simulated mound system consisting of 60 cm of sand and sandy loam over 30 cm of silt loam. Loading consisted of 2 cms of sewage effluent applied four times daily. Mathur and Gruval (1972) conducted a laboratory experiment in which bacteria were injected into a soil medium and samples collected at distances vertically and horizontally from the point of injection. It was reported that 99.9% removal occurred in groundwater after a travel distance of only 15" horizontally and 17" vertically.

Though most studies have been conducted in areas unsuitable for standard soil treatment systems, it is generally agreed that an adequately sited, sized and maintained soil treatment system is capable of preventing groundwater contamination by bacteria (McGauhey, 1975). Saturated soil conditions, (such as might be caused by excessive loading rates, high groundwater tables or impermeable layers), highly permeable soils, and shallow soils over fractured bedrock are all conducive to bacterial travel and contamination of groundwater. However, contamination of deep groundwater tables in slowly permeable soil is unlikely (Bouma et al., 1973).

Virus

Viruses are obligate intracellular parasites ranging in size from .02 um (200A) to .26 um (2600A). In comparison, the smallest bacterium is about .30 um (3000A) in size (Cookson, 1974). Enteric virus concentrations in raw sewage may range up to 1000 infectious units per 100 mls (Sobsey, 1975).

Information concerning virus survival and migration in soil is limited. They appear to survive at least as long as the enteric bacteria (Gerba et al., 1976) and may migrate long distances to contaminate drinking water supplies (Sternberg, 1977).

The main mechanism for removal of viruses appears to be adsorption onto negatively charged soil particles (Gerba et al., 1976 and Wiebel et al., 1964). Bacterial enzyme attack and natural dieoff are also responsible for virus removal (Sproul, 1975). Factors which may influence the removal efficiency of viruses by soil include the following (Gerba et al., 1976 and Drewry et al., 1967).

Flow rate: As flow rate increases, virus rentention decreases proportionally.

Cations: Cations, especially divalent cations, can act to neutralize or reduce repulsive electrostatic potential between negatively charged virus and soil particles, allowing adsorption to proceed.

Clays: This is the most active fraction of the soil. High virus retention by clays results from their high ion exchange capacity and large surface area per volume.

Soluble organics: Soluble organic matter has been shown to compete with viruses for adsorption sites on the soil particles, resulting in decreased adsorption or elution of an already adsorbed virus.

pH: The hydrogen ion concentration has a strong influence on virus stability as well as adsorption and elution. Generally a low pH favors virus adsoption, while a high pH results in elution of adsorbed virus.

Isoelectric point of virus: The optimum pH for virus adsorption is expected to occur at or below its isoelectric point, where the virus possesses no charge or a positive charge. A corresponding negative charge on a soil particle at the same pH would be expected to favor adsorption.

Chemical composition of soil: Certain metal complexes such as magnetic iron oxide have been found to readily adsorb viruses to their surfaces.

Sproul (1975) also reported the following additional factors which may affect virus removal in soil.

Soil Type: Deep soils are most desirable; sandy soils are effective but require greater depth. Factured bedrock provides no removal.

Soil depth: Minimum of 5-10 feet to fractured bedrock or impervious material.

Degree of saturation: Minimum of 5 feet to groundwater table.

Application rate of effluent: Maximum rate of 0.4 to 0.7 gpd/ft^2.

Although very few field studies have been conducted on viruses, Wellings et al., (1974) isolated Polio and Echo viruses in underdrains from wastewater spray irrigation. Both virus types had survived aeration, sunlight exposure, and percolation throught 5 feet of sandy soil and were isolated five months later at 10 and 20 foot depths. In another study, polio virus was also isolated from a well located over 300 feet from any sewage treatment system, although an actual means of intoduction into the well was not identified (Mack et al., 1972).

Other studies, however, show conflicting results. Robeck, et al., 1960 reported studies in which 100% of polio virus were removed from solution by passing effluent through two feet of clean sand with pore sizes 100 to 1000 times larger than the virus particles. Drewry (1969) concluded that virus movement through a continuous stratum of soil under gravity flow conditions, with intermittent dosing should present no health hazards if usual public health practices for well locations are enforced.

The adsorption process responsible for virus removal is effected by many soil and flow regime properties. Though many soils possess the ability to greatly reduce virus concentrations, even one virus is potentially infective and should be of concern.

Nitrogen

Nitrogen forms present in septic tank effluent include: organic nitrogen, nitrate (NO_3), nitrite (NO_2), ammonia (NH_3) and ammonium (NH_4^+). Ammonium and organic nitrogen are the predominant forms, with about 85% to 95% in the ammonium form (Lance, 1972 and Bouma, 1972).

Once in the soil environment, several mechanisms exist through which nitrogen can be removed (Lance, 1972, Sikora et al., 1976). These mechanisms include, volatization of ammonia, vegetative uptake, incorporation by soil microbes, denitrification, and soil adsorption of ammonium.

Volatization of ammonium to ammonia ($NH_4^+ + OH \rightarrow NH_3 + H_2O$) occurs only to a slight degree in septic tanks. At the normal septic tank effluent pH of 7.5-8.0 less than 10% of the ammonium is volatized. Lack of sufficient air-water contact also minimizes volatization to ammonia. (Sikora et al., 1976).

Removal of nitrogen by vegetative uptake depends upon plant roots extending into the effluent-laden layer of soil. Depth of root penetration and proximity of the moisture to the surface are important factors.

An important factor in the movement of water through unsaturated soil is soil moisture tension. This factor involves adhesive forces between water and soil particles and cohesive forces between water molecules. The result, a force very similar to capillary forces, causes water to be drawn into the pore space of soils, even against the force of gravity. Like capillary forces, larger soil moisture tensions are created in smaller pores. Thus, finer textured soils may draw water several inches against the force of gravity. This phenomenon occurs only under unsaturated conditions. Under saturated conditions, the weight of water becomes a dominant factor, pushing the soil water downward. Keppner (1972) reports that water intoduced into clay and loam soils tends to move upward and laterally as opposed to the downward and lateral movement seen in sandy soils. In very fine soils, this upward movement can result in a significant part of the effluent being brought into reach of plant roots.

Plant uptake is generally capable of removing only a relatively small portion of the total available nitrogen, since nitrogen availability far exceeds plant requirements (Sikora et al., 1976). If plants are not harvested and removed, nitrogen is merely recycled back to the soil. Incorporation of nitrogen into soil microbes also removes only a small percentage (5-10%) of the total available nitrogen (Lance, 1972).

Denitrification, the most desirable means of removing nitrogen, is rate dependent on pH and temperature (Lance, 1972). The conditions required for denitrification include: (1) nitrification of NH+ under aerobic conditions; (2) subsequent anaerobic conditions under which nitrate is converted by denitrifying bacteria to various forms of nitrogen gas (N_2,N_2O, and NO); and (3) presence of a carbon energy source for the denitrifying bacteria. However, these concurrent conditions are rarely present to any degree in the soil-adsorption field environment. Sikora et al., (1974) reported that use of Thiobacillus denitrificans for denitrification appeared limited due to high sulfate concentrations obtained during laboratory studies.

Generally, major ammonium (NH_4^+) removal in soil occurs either through anaerobic soil adsorption and fixation or aerobic conversion to nitrate with subsequent leaching. Sikora et al., (1976) reported that the primary factors determining the extent of NH_4^+ adsorption are: the number of soil cation exchange sites exposed to the percolating effluent, the affinity of the sites to NH_4^+, the degree of exchange site saturation with NH_4^+, and the composition of the effluent. Polta (1969) found that NH_4^+ adsorption ranged from 2 mg N per 100 grams of sandy soil to 10 mg N per 100 grams of fine grained soil with 30% clay. As exchange sites become filled, NH_4^+ ions in the percolating effluent proceed to areas of the soil where free adsorption sites were available. Ammonium has been observed to travel in this manner 5 cms to 100 cms per year in various soils (Brown et al., 1977).

Ammonium pollution of groundwater has been reported in soils with low cation exchange capacities (e.g., sands) and highly permeable soils (e.g., glacial tills) with groundwater tables less than five feet (Dudley et al., 1973).

Walker et al published the results of a study of four different drainfields in 1973. One of the drainfields was completely submerged in groundwater. The predominant species in sampling wells was ammonium, which attenuated rapidly with distance from the drainfield (from approximately 65 mg/l to approximately 5 mg/l in about 15 feet). It was concluded that ammonium was predominant as the result of the saturated soil and that attenuation was due to adsorption (Walker et al. 1973).

Nitrification ($NH_4^+ + 2O_2 \xrightarrow[\text{Nitrosomonas}]{} NO_2^- + 2H_2O$ followed by $2NO_2^- + O_2 \xrightarrow[\text{Nitrobacter}]{} 2NO_3^-$) primarily occurs in the initial two feet of soil (Preul et al., 1968) and proceeds to a fairly complete degree when high groundwater is absent (Bouma, 1972; Viraraghaven et al., 1976a). Nitrite (NO_2^-) does not accumulate, since the conversion from nitrite to nitrate ($NO_2^- \rightarrow NO_3^-$) is more rapid than is the conversion from ammonium to nitrite ($NH_4^+ \rightarrow NO_2^-$). Nitrate is very soluble and chemically inactive under aerobic conditions. It travels through the soil-water environment with practically no inhibition, and, unless anaerobic denitrification occurs, will not undergo further transformations once in the groundwater (Hall, 1975; Preul et al., 1968).

Soil retention of nitrate in groundwater at normal pH values is reportedly insignificant. Most studies report attenuation by dilution only. However, Preul et al., (1968) suggested that in groundwater with slightly acid pH (pH 6), adsorption may become significant. Preul et. al. (1968) further reported NO_3 attenuations in the order of .2 mg/l (NO_3-N) per foot-travel in groundwater but did not specify the attenuating mechanism.

Several authors also report that the highest concentrations of nitrate and other soluble pollutants are generally found within the top 1.5 meters of the groundwater table (Dudley et al., 1973; Sikora et al., 1976, Walker et al., 1973), indicating that very little turbulent flow mixing takes place.

In summary, ammonium nitrogen (NH_4^+-N) may enter groundwater aquifers through sand or glacial till soil types in the presence of high groundwater. Under most anaerobic soil conditions, NH_4 travels at relatively slow rates through the soil horizon as adsorption sites in the soil are filled. Once in the nitrate form, however, nitrogen migrates freely through soils with percolating effluents.

Phosphorus

In contrast to the highly mobile nitrate nitrogen ion, most phosphorous-bearing compounds react vigorously with soils. It is the major anion to exhibit such behavior in the usually negatively charged soil environment (Preul, 1968).

The principle forms of phosphorus in septic tank effluent are the orthophosphates and polyphosphates (which may be hydrolyzed to orthophosphate). Organically bound phosphates are not common in domestic wastewater (Metcalf and Eddy, 1972) and 70-85% of the phosphate in the average septic tank occurs as soluble orthophosphate ($PO_4^{=}$, $HPO_3^{=}$, and H_2PO_4) (Bouma, 1972 and Otis et al., 1975).

Most soils are capable of adsorbing in excess of several hundred pounds of phosphate per acre foot (Sternberg, 1977), and up to 8000 pounds phosphate adsorption per acre (to a depth of 6 inches) has been reported (Polta, 1968). While phosphate removal mechanisms appear complicated and are not totally understood, it is known that most forms of phosphate are retained in the soil through a combination of adsorption reactions and replacement reactions involving changes in mineral crystalline structure. Also, precipitation reactions resulting in insoluble compounds with Fe, Al, and Ca (Laak et al., 1974; Hall, 1975; and Polta, 1969) may occur in acetic pH ranges. Under alkali conditions, phosphate reacts with calcuim to form an insoluble precipitate (Buckman, 1979). Thus phosphate may be removed in soils at practically all pH ranges. Soluble orthophosphate may also be converted to insoluble forms by certain bacteria (Dudley et al., 1973).

Phosphate ions, like ammonium ions, advance in soil as adsorption sites are occupied (Sawhey, 1977; Magdoff et al., 1974; and Dudley et al., 1973). Sawhney (1977) states that the rate of phosphate movement depends on application rate, soil percolation rate and pH of the soil; though Hall (1975) observed no pH effect on adsorption. Many soils regenerate phosphorous adsorption sites, causing soil adsorption capacity to be greater than determined by equilibrium measurements made in the laboratory. This regenerative ability has been attributed to the slow precipitation reaction mechanism (Sawhney et al., 1975: and Sawhney et al., 1977).

For up to three years after its initial adsorption, phosphate which has been removed and held by the soil can be released when the soil is suspended in water (Kuntz, 1946 and Sawhney et al., 1975). Hill (1972) also reported that a fluctuating water table may disturb the phosphate equilibrium, causing phosphate to go into solution and be transported. While Ellis et al., (1973) reported data which suggests that phosphate adsorption may occur below the water table, other studies report travel of water-borne phosphate in excess of 100 feet (Dudley et al., 1973 and Ellis et al., 1973). Dudley et al., (1973) reported up to 7.5 mg/l total phosphate reaching groundwater at 20 feet after percolation through well-drained sandy soil, while Bouma et al., (1972) reported low phosphorus concentrations in groundwater after percolation through sand.

Though many soils are capable of reasonably efficient removal of phosphate, clean sandy soils lacking Fe or Al oxides, soils with high groundwater, shallow soils over fractured bedrock, or soils around old soil-adsorption systems often provide inadequate phosphate removals (Sikora et al., 1976; Stenberg, 1977; Walker et al., 1973; and Viraraghaven et al., 1976a). Systems with high loading rates usually do not remove phosphate well.

Biodegradable Material

Biodegradable organic material is also found in septic tank waste, as can be seen in Table 1. This material is usually removed in the soil by physical straining and by adhesion to soil particles. Once removed, it is utilized by soil bacteria as a food source and eventually decomposed.

Not much attention has been given to biodegradable organics in previous research on septic tanks. Only one study reviewed presented data on BOD, COD, and organic carbon. Viraraghavan and Warnock (1976b) investigated concentrations of BOD, COD, and total (soluble) organic carbon beneath drainfields. Ranges of values reported are as follows:

BOD	21 - 560
COD	o - 218
TOC	5 - 54

These values attenuated rapidly to background concentrations in approximately 10 ft.

Other Constituents

The average per capita contribution of chloride to wastewater is six grams per day (Metcalf and Eddy, 1972). Chloride is very soluble and mobile in the soil-water environment and is not exchanged or adsorbed (Schmidt, 1972). Since nitrate and chloride are both very mobile and are both present in septic tank effluents, a correlation between the two is often used to document septic system pollution of groundwater supplies (Hill, 1972; Dudley et al., 1973; and Schmidt, 1972).

Hill (1972) reported that six representative Connecticut soils removed very little sulfate and sodium and that leaching sulfate through 100-cm soil columns removed up to 90% of the sulfate during the first year, but only 40% and zero percent after one and two years respectively.

Heavy metals have been found to accumulate in soils immediately adjacent to treatment fields. However, accumulated concentrations were below those which would restrict plant growth and should not cause excessive degradation of leachate water (Brown et al., 1977). Sandhu (1976) found arsenic was ubiquitous in 468 well samples from three South Carolina counties with 90% of the samples containing arsenic and 24% exceeding the USPHS (1962) mandatory limit of 0.05 mg/l. The data implicates septic systems as a possible source of arsenic, since its presence was directly related to septic system distance from the water supply.

Thomas et al., (1970) reported alkyl benzene sulforate (ABS) concentration values to 32 mg/l in well supplies of Long Island, thus exceeding the USPHS recommended limit of 0.5 mg/l. With decreased use of ABS in the new detergents, this should be less of a problem, though Methylene Blue Active Substances (MBAS) from detergents have recently been reported to exceed the USPHS recommended limits of 0.5 mg/l MBAS.

In summary, even when septic systems are properly sited, installed and maintained according to public health requirements, nitrates, chlorides, and apparently sulfates, ABS, MBAS, and arsenic move freely through the soil with percolating water (Brown et al., 1977; Sandhu et al., 1976; and Hill 1972).

FAILURE OF SOIL ADSORPTION SYSTEMS

As seen in the previous discussion, soil adsorption systems can be very effective in removing contaminants from wastewater under favorable conditions. Unfavorable conditions can decrease the effectiveness of the soil, however, and under extreme conditions, total failure can occur. Failure can result from hydraulic conditions, from inadequate soil characteristics, or, most often, from a combination of the two.

Historically, soil-adsorption systems have been designed on hydraulic parameters. Percolation tests are run to estimate the rate of transfer of water away from the soil adsorption field. Estimated flows are usually based on the number of bedrooms in the residences to be served. Both of these estimating techniques have been shown to be questionable (Bouma, 1975; Miller, 1972; Thomas, et al., 1966). Significant changes in hydraulic conductivity of soils occur due to clogging of the wastewater-soil interface with a biological slime mat, or "crust", which reduces infiltration rates. (Thomas, et al., 1969; Laak et al., 1974; Kropf et al., 1975;

Bouma, 1972) The infiltration rate through this crust is independent of the infiltration rate of the surrounding soil (Kropf et al., 1977) and stabilizes 100 to 200 days after initial use begins (Bouma, 1972).

Fine textured soils are very efficient treatment systems but have low hydraulic conductivity. The problem is compounded when swelling clays are encountered which absorb water into this molecular structure, thus increasing particle size and squeezing pore space closed. Soil adsorption systems which are hydraulically overloaded fail when the rate of wastewater application exceeds the percolation rate and the soil becomes saturated. As noted previously, adsorption mechanisms do not function efficiently under saturated conditions. Additionally, saturation greatly reduces the air-water interface and anaerobic conditions prevail, preventing microbial oxidation. Improperly treated wastewater may then move into the groundwater. This condition is especially undesirable when high groundwater tables are underneath the soil adsorption system.

Hydraulic failure can also result from heavy loading on highly permeable soils. Porous soils are not as effective at screening out bacteria and have less surface area on which adsorption can occur. Although aerobic conditions usually prevail, the contact time necessary for biological processes to be effective may not be maintained at high loading rates. An extreme condition would prevail where coarse, shallow soils overlay fissured bedrock aquifers (Allen & Morrison, 1973).

In addition to hydraulic and soil failures, other features of the adsorption site are important. From the standpoint of groundwater pollution, the most important is the vertical distance to the aquifer. Even low hydraulic loading on good soils will result in contaminants reaching the groundwater if sufficient soil depth is not provided for adequate treatment.

EFFECTS OF FAILURES ON GROUNDWATER QUALITY

Several studies have shown that under various soil-water regimes, pollution of groundwater aquifers occurs on a localized scale as a result of subsurface wastewater disposal. More recent studies have shown that in some areas where numerous septic systems have been installed, large scale pollution of major groundwater aquifers has occurred.

Miller (1972) conducted two studies in the coastal plain of Delaware where homes were located on one-quarter to one-half acre lots. Both areas covered less than one square mile. In the first study, high groundwater tables and slowly permeable soils perdominated. Fifty-five percent of the area was classified in 1970 by the U.S. Department of Agriculture as having severe septic tank limitations, and overflowing septic tanks and odors were reported. Miller found that most wells at this site had nitrate levels below 20 mg/l (as NO_3). At the second study site, low groundwater table and well-drained soil conditions predominated. The U.S. Department of Agriculture rated the soil as having only slight limitations for septic systems. Nitrate levels at this site ranged from 22 mg/l to 136 mg/l (as NO_3) with many wells exceeding the 45 mg/l (as NO_3) set by the EPA interim standards. Obviously, USDA appraisal of soil suitability was based on hydraulic failure and not treatment failure potentials.

In a report by Woodward et al., (1961), data showed that 48% (30,000) of the 25 to 40 foot wells sampled in 39 Minnesota communities were contaminated with nitrate and surfactants. About 11% (6,740) of the wells contained nitrate in excess of 45 mg/l (as NO_3). Though the findings varied considerably from community to community, up to 50% of the wells showing nitrate contamination were also contaminated with coliform bacteria. The factors reported as influencing contamination included well depth, population density, character of soil, age of the systems, and direction and speed of goundwater movement.

Schmidt (1972) attributed groundwater pollution by nitrates in excess of 45 mg/l (as NO_3) in the Fresno-Clovis area of California to shallow wells and septic systems located on small lots. Chlorides and nitrate concentrations showed high correlation in this study.

Quan et al., (1974) reported groundwater contamination from individual sewage treatment systems around Portland, Oregon. Wells in an unsewered area had nitrate concentrations of about 22 mg/l (as NO_3), however, chloride concentrations were low, suggesting other sources may also have been involved.

Septic system effluent was reported (U.S. EPA, 1977) to be the major contributor to signficant contamination of a major aquifer in an 80-square-mile area of Nassau County, Long Island. Nitrates penetrated hundreds of feet into the principle artesian aquifer, degrading beyond the EPA interim standards 16 public water supplies which served thousands of people.

Drewry et al., (1968) reviewed 664 reports of waterborne disease outbreaks and reported that in most cases where viral or bacterial agents appeared to have traveled in groundwater, evidence indicated movement was through fissured or fractured substrata, rather than the soil itself. Waltz (1972) found that 62% of the wells from 35 mountain homesites were contaminated with coliform bacteria. Presumably travel was through fractured bedrock.

Miller (1977) cited an incident in Mechlenburg County, North Carolina, in which a well polluted by septic system effluent caused over 500 cases of shigellosis in 1000 people using the well. The well was properly constructed and overland flow was ruled out.

Morrill et al., (1973) reported results from a study conducted in the Ipswich and Shawshee river basins of Massachusetts indicating that some housing developments have lowered the quality of the goundwater environment and have created health hazards. Housing densities ranged up to 900 units per square mile (three-quarter-acre lots).

In addition to the studies discussed here, a number of other case studies can be found in the U.S. EPA's series of regional groundwater studies (Van der Leeden et al., 1975; and Miller et al., 1977).

THE MONTANA PROJECT

The Montana study was designed to determine the effects of unsewered subdivision on groundwater quality. Like many of the studies reported in the previous section, the sites investigated represented extreme conditions which presented the greatest potential for groundwater pollution. The principal criterion for selection was porous soil over high ground water. The sponsoring agency selected the subdivisions for investigation. In all, five subdivisions were studied. The approximate location of these subdivisions are shown on the location map on the back cover of this report.

Site Description

Hawker Lane Estates, located North of Hamilton, Montana is a relatively isolated subdivision consisting of 27 lots of about one acre each. The population density is approximately 2.0 persons per acre with 16 of the lots developed. High seasonal ground water varies between 2.3 and 5.3 feet below the ground's surface and is probably influenced most strongly by Grid Creek which flows through the western half of the subdivision. Ground water flows in a Northwesterly direction. Three well points (averaging 7.5 feet in depth) were used to sample the upper strata of the water table, one located at the upstream end and two at the downstream end of the subdivision. Four residential wells (averaging about 35 feet in depth) located along the flow gradient, were also sampled. The relative location of these sampling points are shown on Map No. One attached to the back cover of this report.

Lambkin Addition, located East of Lincoln, Montana consists of the Lincoln School and about 200 lots averaging 10,000 square feet. With 40 homes in the subdivision, the current population density is approximately 2.7 people per acre. The potential population density in the subdivision when fully developed is about 14.2 people per acres. High seasonal groundwater in the subdivision varies from 1.2 to 5 feet below the ground's surface, and flow is in a Southwesterly direction. Four well points (averaging 7.5 feet in depth) were located along the groundwater flow gradient passing through the subdivision. Five residential wells, averaging 29 feet in depth, were also sampled. Location of these sampling points are shown on Map No. Two located on the back cover.

Five Acre Tracts, located near Livingston, Montana, consists of approximately 285 lots. Approximately 32 percent of the lots are less than one acre, 33 percent are between one and two and one-half acres, and 35% average about five acres in size. The subdivision is about 30 percent developed with a population density of roughly one person per acre. Development trends have, however, created areas of higher density. High seasonal groundwater varies from less than 1.7 feet to over 11 feet and is strongly associated with irrigation practices. Groundwater flow is generally in a Northeasterly direction. Three well points (averaging 8 feet in depth) were used to sample the upper levels of the water table and seven residential wells (averaging 26 feet in depth) for sampling lower levels of the aquifer. Sampling sites were dispersed along the groundwater flow gradient passing through the subdivision. Sampling sites are shown on Map No. Three attached to the back cover.

Entropy Acres, area east of Hamilton, Montana, includes Entropy acres, Big Sky Subdivision, and a portion of Daly Subdivision containing a total of 55 platted lots, each averaging approximately 40,000 square feet in size. The current population density is about 1.8 people per acre with about 70 percent of the lots developed. High seasonal groundwater in the area varies from 2.3 to 7.0 feet and is strongly influenced by local irrigation practices. Groundwater flow is in a Northwesterly direction. Sampling sites consisted of four well points (averaging 8.5 feet in depth) and four residential wells (averaging 40 feet in depth) dispersed along the upper to lower groundwater flow gradient passing through the subdivision. Location of the sampling points are shown in Map No. Four attached to the back cover.

Sewell Subdivision #4, located North of Helena, Montana, consists of 82 one-half acre lots of which 63 percent are developed. Population density is about 5.4 people per acre. High seasonal ground water in the subdivision is as high as 1.4 feet below the ground's surface and is influenced strongly by local irrigation practices and Spring Creek. Sampling sites consisted of four well points (averaging 7.5 feet in depth) and four residential wells (averaging 45 feet in depth) which were dispersed along the groundwater flow gradient passing through the subdivision. Location of the sampling points are shown on Map No. Five attached to the back cover.

Soil conditions found in the subdivisions are delineated in a soils map located on the back cover of this text.

Installation of Sampling Points

Sampling points were established in the upper groundwater levels by driving well points into the ground. The well points were 1-1/2" in diameter and were attached by a threaded collar to a steel pipe of the same diameter. They were driven into the ground manually using a post driver. Since it was desired to collect samples from the top of the ground water, well points were sunk only 1-2 feet below the groundwater table. Since many of these sampling wells were sunk only to the depth to which ground water was reported to rise during high groundwater season. This presented problems later when ground water failed to rise to the anticipated level at some locations.

The installation of the well points presented some difficulty in areas underlain by large gravel and cobbles. In some instances, heavy gravel points were purchased and installed with considerable difficulty.

Other samples were taken from wells currently in use as domestic water supplies. Generally, these wells were in excess of 30 feet deep and were often separated by an impervious (or semi impervious) strata from the ground water sampled through the well points.

Sampling Techniques

Samples were obtained from the well points with a hand pump and tygon tubing. Before each use, the pump and tubing were flushed by pumping from a container of distilled water just prior to sampling. Prior to taking a sample, enough water was pumped from each well point to removing the water standing in the pipe, and to flush all the distilled water out of pump and tubing.

Samples from the residential water supply wells were taken from hydrants located as close to the well as possible. Prior to sampling enough water was wasted to cause the pump to start up, thus insuring a fresh sample which had not retained in the pressure tank for any significant length of time.

All samples were then taken in bottles of appropriate volumes for the various tests. Samples for bacterial analysis were collected in sterilized bottles while bottles for other tests were cleaned by appropriate means to preserve the integrity of the samples. The bottles of samples were then packed in ice for the return trip to Bozeman.

Well point samples were taken from the top one foot of the groundwater table. Previous studies (Walker, 1973) indicate that little vertical mixing occurs and thus levels of contaminents in groundwater will be greatest in the top layer.

SAMPLING FREQUENCY

The project sampling began in September 1978, and was completed in August 1979. Major sampling efforts were made during the spring and summer months of 1979. Fewer samples were taken during the fall and winter months since groundwater levels were low and weather conditions were extreme. Entropy acres, selected while the study was in progress, was not sampled during the early stages of the project. The following table lists the sampling dates for each subdivision.

TABLE 2. SAMPLING FREQUENCY

Five Acre Tracts	Entropy Acres	Hawker Lane Estates	Lambkin Addition	Sewell Subdivision No. 4
9/26/78				
10/26/78			10/27/78	10/27/78
		11/18/78		
			1/13/79	1/13/79
2/09/79				
3/23/79	3/21/79	3/21/79		
4/21/79			4/21/79	4/21/79
6/14/79	6/02/79	6/02/79	6/14/79	6/14/79
	6/30/79	6/30/79		
7/14/79	7/13/79	7/13/79	7/13/79	7/13/79
7/27/79	7/28/79	7/28/79	7/28/79	7/28/79
8/17/79	8/18/79	8/18/79	8/19/79	8/19/79

Analysis

The samples were analysed for pH, specific conductance, chlorides, nitrates, ammonia, phosphates, chemical oxygen demand, and feccel caliform bacteria. All analysis were run according to procedures described in Standard Methods for Examination of Water and Wastewater, 14th Edition, or Methods of Chemical Analysis of Water and Waste (EPA 1974), or equivalent procedures. Samples were either tested within the prescribed time or were appropriately preserved for later analysis.

All of the data obtained in this study is listed by sites and date in the Appendix of this report. Average values for each parameter, except pH, at each site is given in Table 3 on the following page. Since pH values are logrithm functions, they should not be averaged in the usual manner. A discussion of the parameters and data is included in the following sections of this report.

pH

The pH of water is a measure of the molar concentration of hydrogen ions in solution. Mathematically, the pH is expressed as the negative log of the hydrogen ion concentration in moles per liter. The pH of ground water is an important parameter since it plays a vital role in the ammonia-ammonium equilibrium, and in adsorption capacity of the soil for several pollutant species.

Since pH values of ground water may change if the water is exposed to air, pH analysis was performed in the field immediately after sampling. A portable pH meter (Orion Research specific ion meter Model 404) was used for this purpose.

An examination of the pH data in the Appendix reveals values very near neutral for all sites. No significant difference can be noted between upstream control sites and sites within the subdivision.

Specific Conductance

The ability of water to conduct electricity is a function of its ionic strength. Thus, a measurement of the electrical conductivity is

TABLE AVERAGE PARAMETER VALUES

	Parameter							
Site	Specific Conductance micromhos/cm	Fecal Coliforms per 100 mls	Cl mg/l	NO_3^-N mg/l	NH_3^-N mg/l	COD mg/l	PO_4P mg/l	Groundwater Depth ft.
Hawker Lane Estate								
HM-1P	186	<1.14	1.8	.3	0.02	8.0	0.04	4.5
HM-2W	244	<1	2.3	.4	<0.01	.6	0.13	NA*
HM-3P	178	<1	2.4	.4	0.03	11.4	0.05	5.4
HM-4W	181	<1	1.9	.3	<0.01	2.6	0.07	NA
HM-5W	203	<1	2.0	.5	1.35	4.7	0.15	NA
HM-6W	254	<1	1.8	.3	0.01	.6	0.02	NA
HM-7P	248	<1	2.0	.2	0.03	4.1	0.10	2.8
Entropy Acres								
HD-1P	416	<1	1.7	.7	2.02	7.0	0.16	3.8
HD-2W	389	<1	1.6	.9	<0.01	1.7	0.08	NA
HD-3P	299	<1	1.4	.6	0.16	9.1	0.09	6.6
HD-4W	381	<1	1.8	.7	0.02	0	0.02	NA
HD-5P	295	<1	1.4	.6	0.02	8.7	0.06	6.2
HD-6W	373	<1	1.6	.6	1.68	1.4	0.05	NA
HD-7P	313	<1	1.4	.6	0.01	6.1	0.09	7.0
HD-8W	383	<1	1.3	.6	0.02	0	0.02	NA
Five Acre Tracts								
LV-1W	632	<1	4.7	.9	0.01	13.5	0.08	NA
LV-2W	341	<1	3.2	1.3	0.31	15.4	0.06	NA
LV-3W	544	<1	8.1	.7	0.01	15.0	0.05	NA
LV-4W	416	<1	5.6	.5	0.05	16.4	0.08	NA
LV-5W	491	<1.17	5.9	.5	<0.01	12.1	0.06	NA
LV-6P	418	<2	1.1	.5	<0.01	13.4	0.06	5.4
LV-7P	269	<1	2.5	.4	0.01	11.6	0.06	2.8
LV-8P	NS							+
LV-9W	NS							
LV-10W	413	<1	3.4	.7	<0.01	21.3	0.05	NA
Lambkin Addition								
LN-1P	350	<1.5	2.5	.3	0.01	17.9	0.03	9.0
LN-2P	366	<1	2.0	1.1	<0.01	13.6	0.06	2.9
LN-3W	357	<1	1.9	.2	<0.01	8.3	0.08	NA
LN-4W	332	<1	2.2	.3	<0.01	0	0.06	NA
LN-5P	364	<1	2.5	.3	0.02	9.6	0.01	5.2
LN-6W	360	<1	1.9	.2	<0.01	6.0	<0.01	NA
LN-7P	390	<1	2.3	.3	<0.01	18.5	0.12	3.1
LN-8W	363	<1	1.9	.3	<0.01	9.1	0.03	NA
LN-9P	421	<1	2.6	.2	6.47	23.4	0.10	1.9
Sewell Subdivision #4								
HV-1W	652	<1	15.1	1.4	0.01	4.1	0.04	NA
HV-2P	685	<1	5.1	.2	0.03	34	0.04	2.2
HV-3W	715	<1	8.4	1.2	0.86	24	0.05	NA
HV-4P	NS							
HV-5W	736	<1	7.1	1.0	0.01	8.5	0.41	NA
HV-6P	NS							
HV-7W	699	<1	7.8	1.4	3.21	4.5	0.08	NA
HV-8P	913	<1	5.9	.2	0.24	0	0.04	1.4

*Not applicable
+No sample

an indirect measurement of the ions in solution, or the total dissolved solids (TDS) of the water. Electrical conductivity is the inverse of electrical resistance and is expressed in units of mhos. Standard procedure is to measure the conductivity of a one cubic centimeter field at 25°C and express the result as micro-mhos (μ mhos) per cm. A conversion factor varying from 0.55 to 0.9, depending on the activities if the ions in solution, is then used to convert the specific conductivity (SC) to TDS expressed as millegrams per liter. For most natural waters, a conversion factor of 0.65 gives good TDS values.

Specific conductance was, therefore, an important parameter in this project since septic tank effluent is expected to have a higher TDS than surrounding ground water. Examination of the data shows a wide variance in the SC of ground water at the different subdivisions, which can be expected. Additionally, samples drawn from water supply wells generally showed higher SC values than samples from well points. This also expected since water in deeper aquifers generally has been in contact with soil longer than water in shallow aquifers, thereby increasing its TDS load.

Although inspection of the data in Table 3 and the Appendix reveals some differences in S.C. values between well points and between water supply wells in each subdivision, no pattern of differences can be noted with any regularity.

Chloride

Chloride analysis was performed by the mercuric nitrate titrimetric procedure using alliquots from refrigerated samples. As previously stated, chlorides move freely through the soil and are apparently attenuated by the groundwater. Like specific conductance, an increase of chloride in the ground water beneath the test sites would indicate contamination from septic drainage.

As seen in the Appendix and Table 3, concentration of chloride was very low in all of the samples analyzed. Concentration differences appear to be more or less randomly distributed throughout the subdivisions.

Nitrate and Ammonia

A thorough discussion of nitrate and ammonia was included in a previous section. Samples for analysis were collected in separate bottles, the bottles were completely filled and placed on ice for transportation back to the laboratory. Analyses were made by the specific ion technique using an Orion Research Digital Ionalyzer Model 801A. Accuracy was periodically checked by the addition of known quantities of nitrate to samples.

Data from nitrate and ammonia analysis generally shows very low values of each. Occasionally, an average value in Table 3 is seen to be an order of magnitude or so above the other values. (e.g., Nitrate at LN-2P, LV-2W, HV-1W, ammonia at HM-5W, LN-9P, HD-1P, HD-6W, HV-7W). A check of individual data points in the Appendix for these sites invariably shows the higher averages to result from one, or at the most two, high values. These "spikes" were noted at the time of analysis and checks made on the samples confirmed the high values. The reason for the spikes are not known.

Even higher values for nitrate and ammonia shown in Table 3 are below the level of concern. Again, differences appear to be randomly distributed throughout the subdivisions.

Chemical Oxygen Demand

The COD test is a measure of the oxidizable material in the water.

Most oxidizable material in septic tank waste should be organic in nature, so COD should give a good indication of biodegradable material. Samples for COD analysis were collected in individual bottles, acidified to prevent biolological degradation, and transported back to the lab on ice. Analysis was made by the rapid COD method outlined by EPA.

Although some variance in the average values of COD, in Table 3 are noted, no significant differences are apparent. Two possible exceptions (HV-2P and HV-3W) can be traced back to one sample from each well with an unusually high COD (sample taken on 6/14/79). In general the COD values shown are not high enough to cause concern.

Phosphate

Samples for phosphate analysis were collected in specially cleaned (no detergent, acid washed) bottles. Analysis was made by the stannous chloride method (Standard Methods) using a Bausch and Lomb Spectronic 20 spectrophotometer.

Concentrations of phosphate in Table 3 are seen to be extremely low. Again, any variances appear to be randomly distributed throughout the test sites.

Fecal Caliform

Fecal Caliform analyses were run using the membrane filter technique. Samples for fecal caliform analyses were taken in sterilized bottles and were run as quickly as possible. Sampling trips were usually arranged so that the two subdivisions at Hamilton, Lambkins Addition at Lincoln, and Sewell Subdivision at Helena were sampled in a two day trip. A vacuum pump and membrane filter equipment were carried and the samples collected at Hamilton were plated out in the evening at the motel in Lincoln. The plates were incubated in a millipore incubator which works on both AC and DC current. House current was used at the motel while the auto electrical system was used during transport to MSU the following day. Samples taken at Lincoln and Helena were packed in ice and run at the MSU lab. Thus, a maximum time of about 12 hours lapsed for samples taken at Lambkins.

Although sampling for coliform analysis was done as carefully as possible, complete sterile conditions could not be maintained when sampling the well points. The pump and tubing used in sampling could not be sterilized prior to each use. It was felt, however, that bacterial sampling could still give useful data since the absence of any positive plate counts would indicate no bacteria present in the ground water. (Positive counts would indicate either bacteria in the ground water or contamination of the pumping equipment). Since only a very few samples showed counts, and since these were very low counts, it is reasonable to assume that the ground water at all sites was essentially free of fecal caliform bacteria.

DATA INTERPRETATION

To illustrate any changes in groundwater quality which could possibly be the result of septic tank effluent, data for both well points and water supply wells were plotted showing concentrations of selected parameters versus relative upstream-downstream positions. (Figure 1 through Figure 10). Changes in values over time are shown in Figures 11-20. Locations of the sampling points are shown on Maps 1-5 on the back cover of this report.

These figures (and Table 3) indicate no significant changes in water quality as it flows beneath the subdivisions. Well points at

Fig. 1 Entropy Acres

Individual Well Point Averages

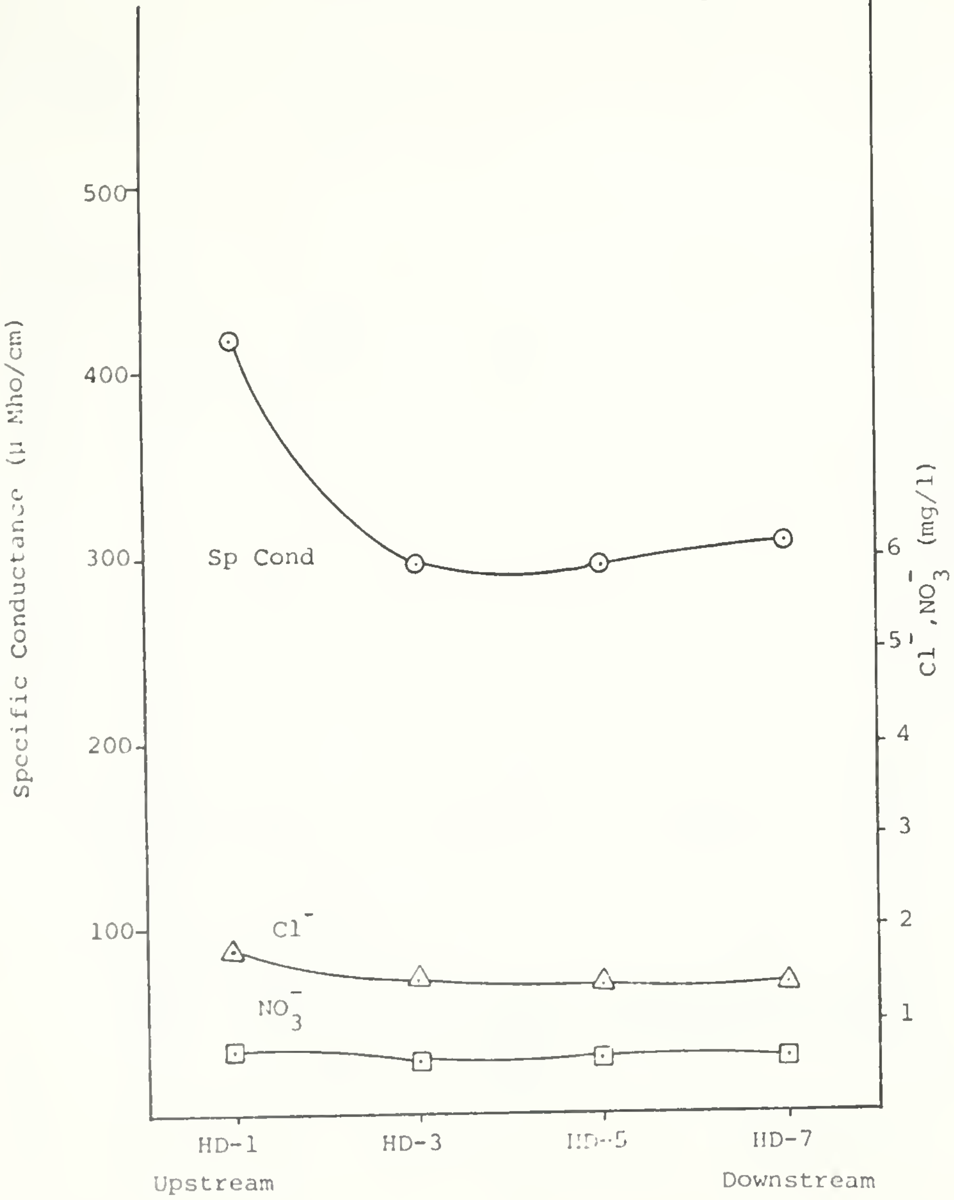


Fig. 2 Entropy Acres

Individual Residential Well Averages

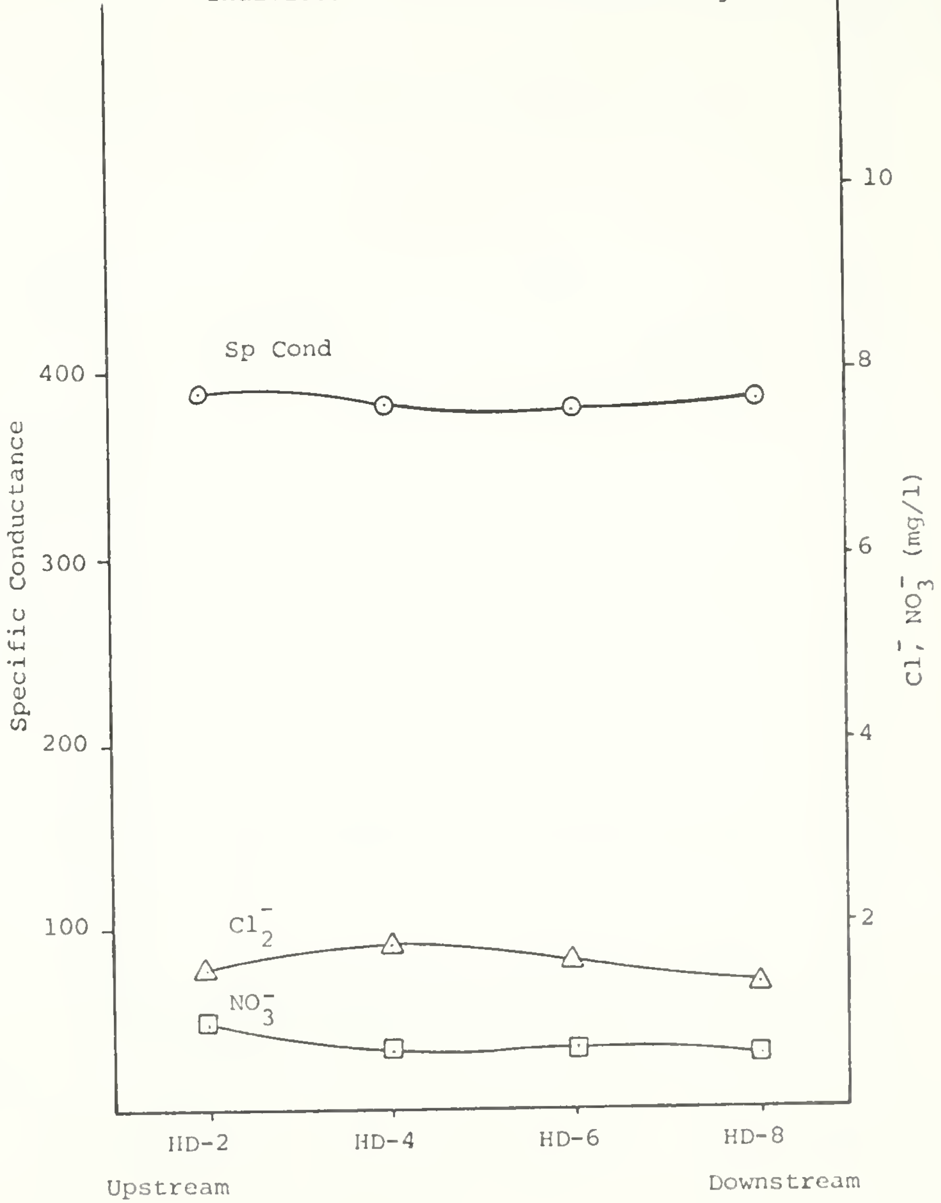


Fig. 3 Hawker Lane

Individual Well Point Averages

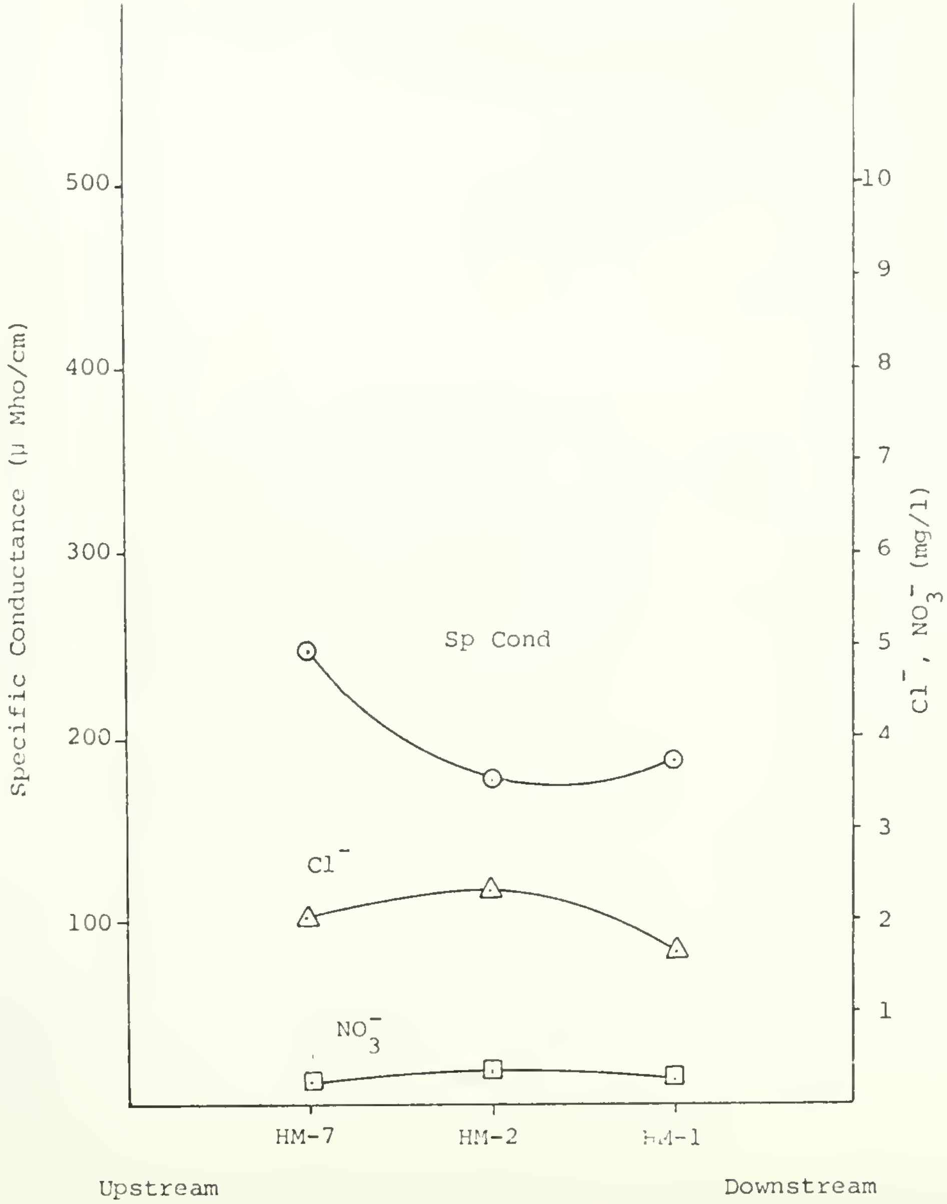


Fig. 4 Hawker Lane

Individual Residential Well Averages

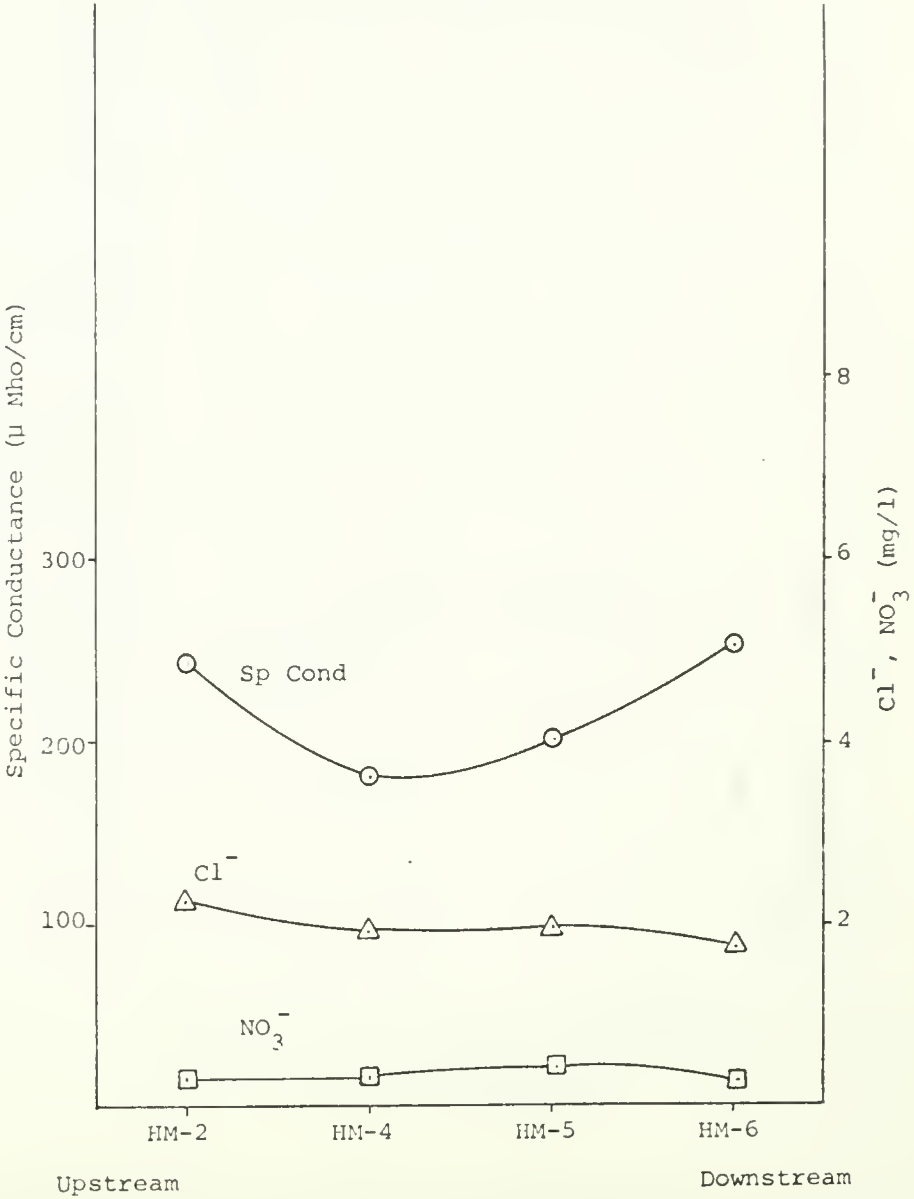


Fig. 5 Lambkins

Individual Well Point Averages

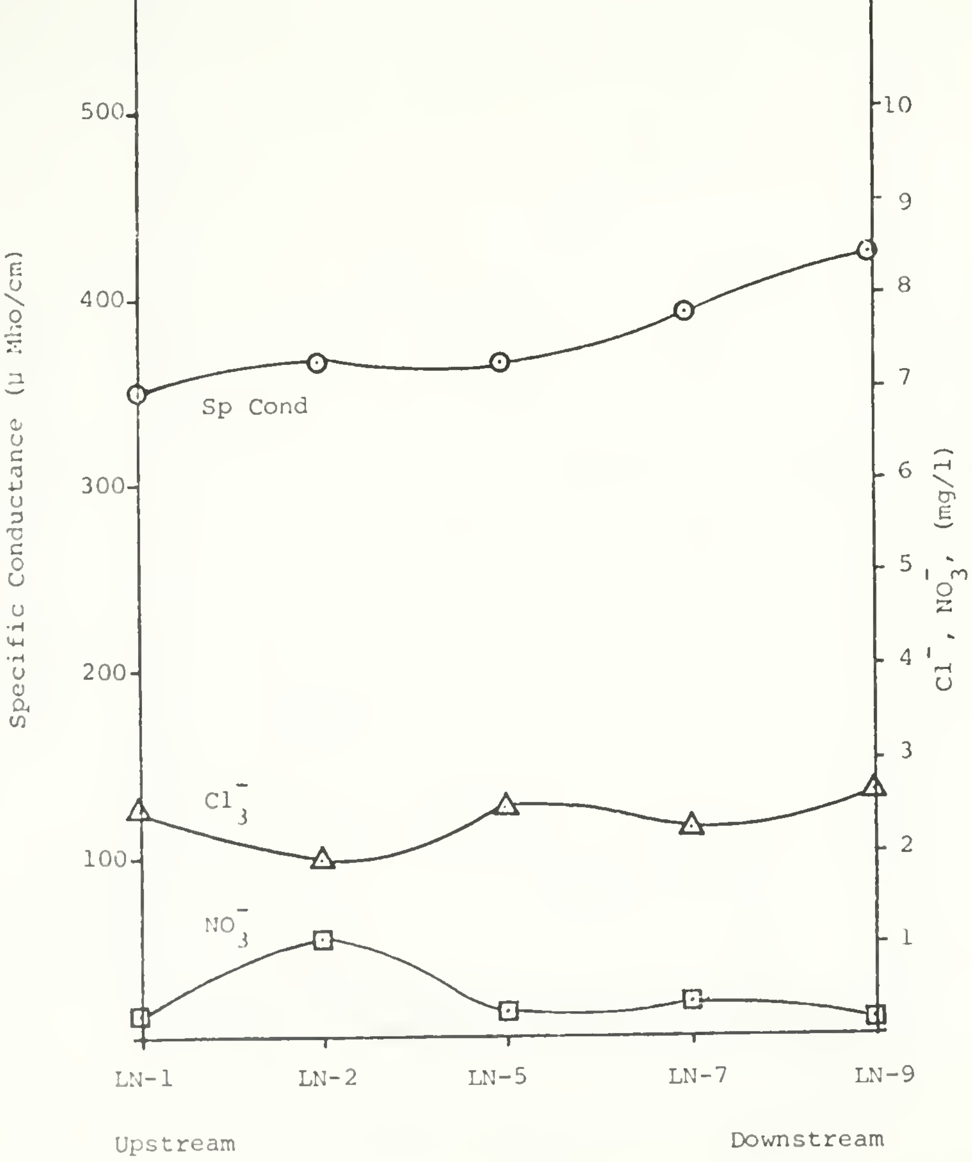
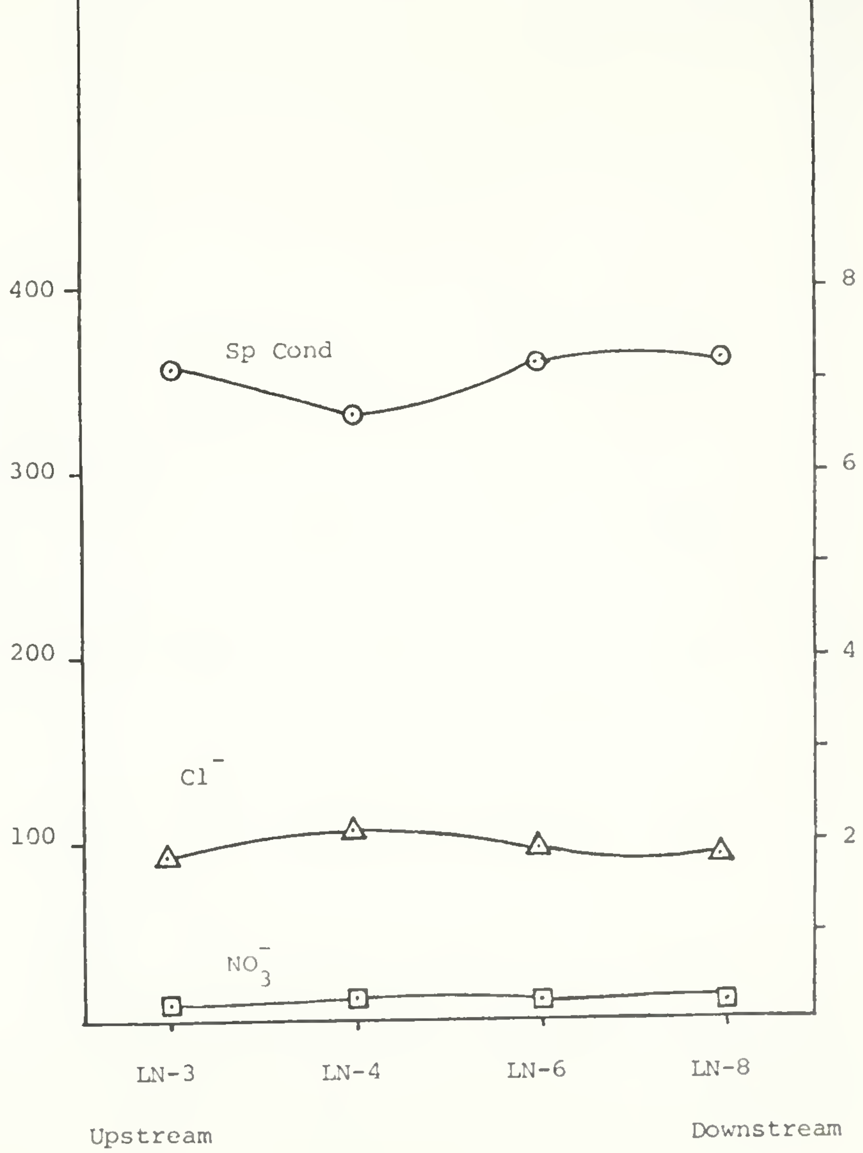


Fig. 6 Lambkins

Individual Residential Well Point Averages



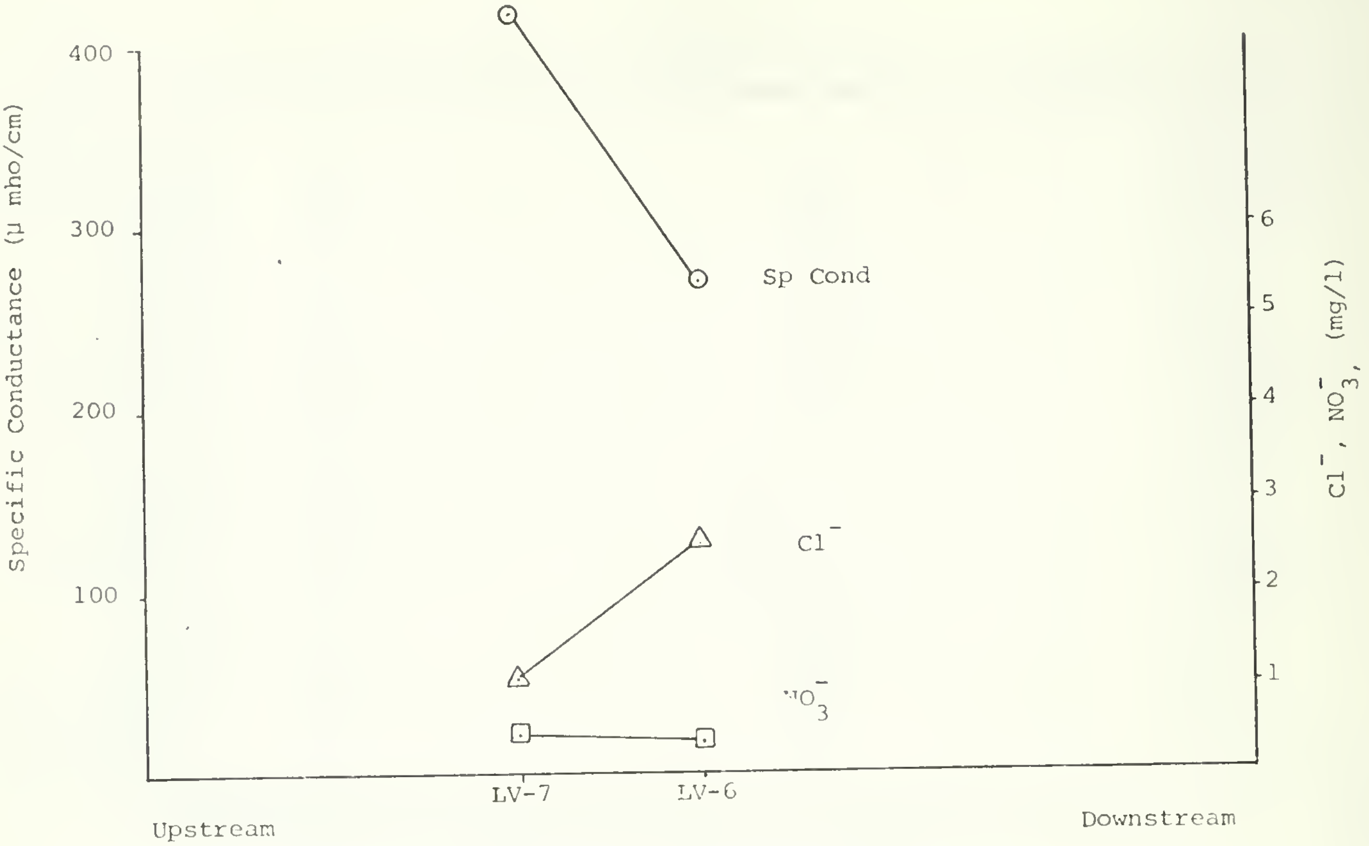


Fig. 7 Livingston
Individual Well Point Averages

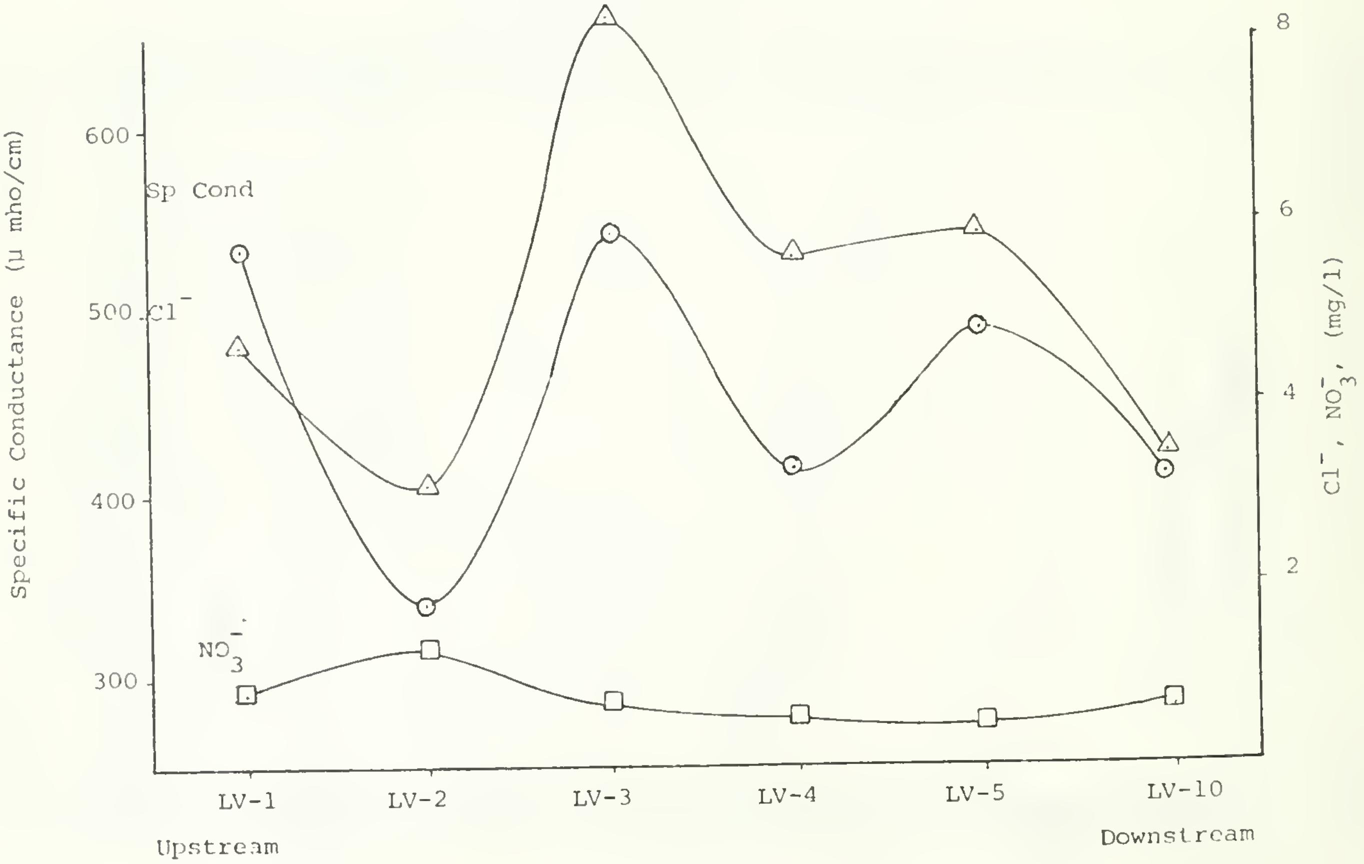
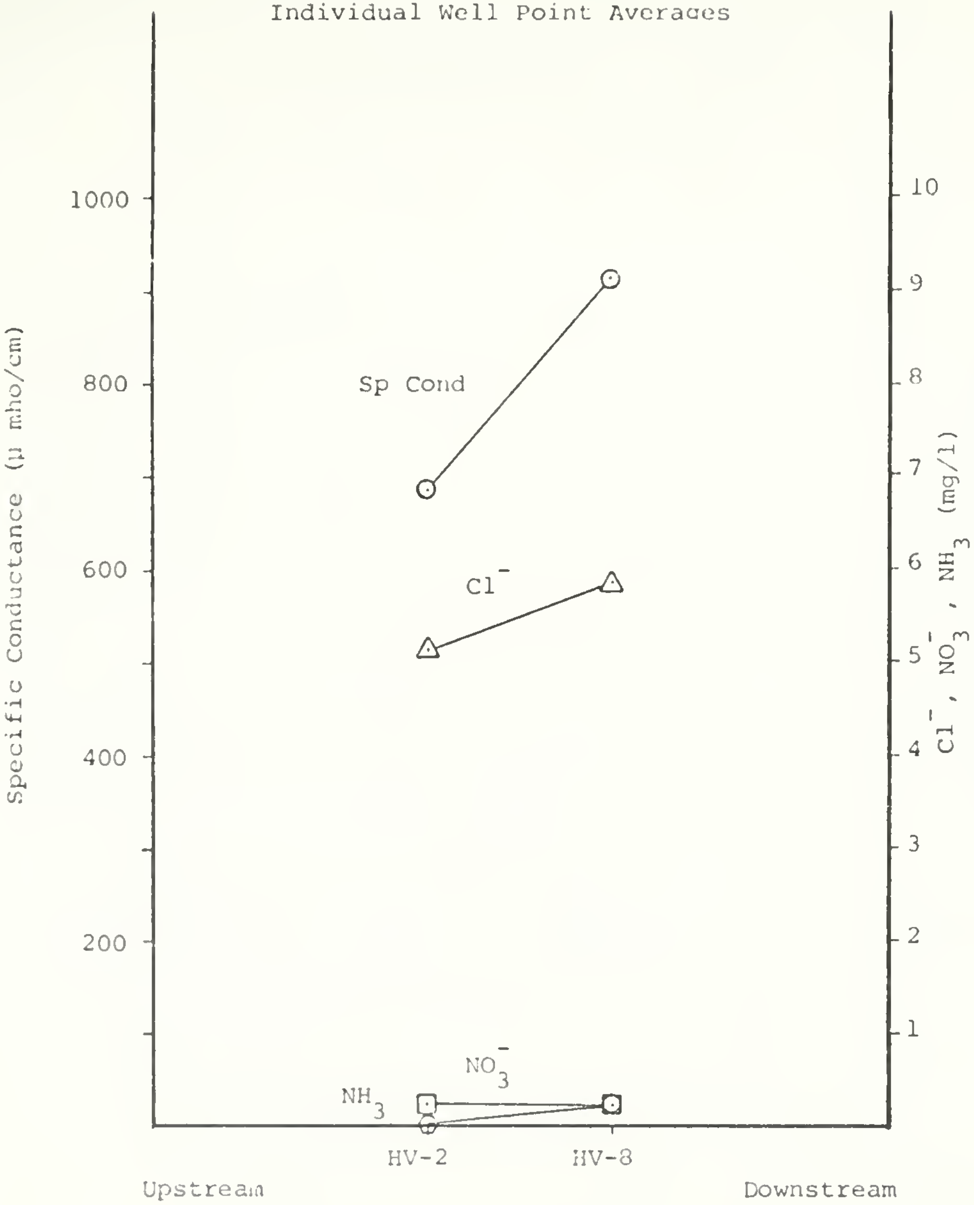


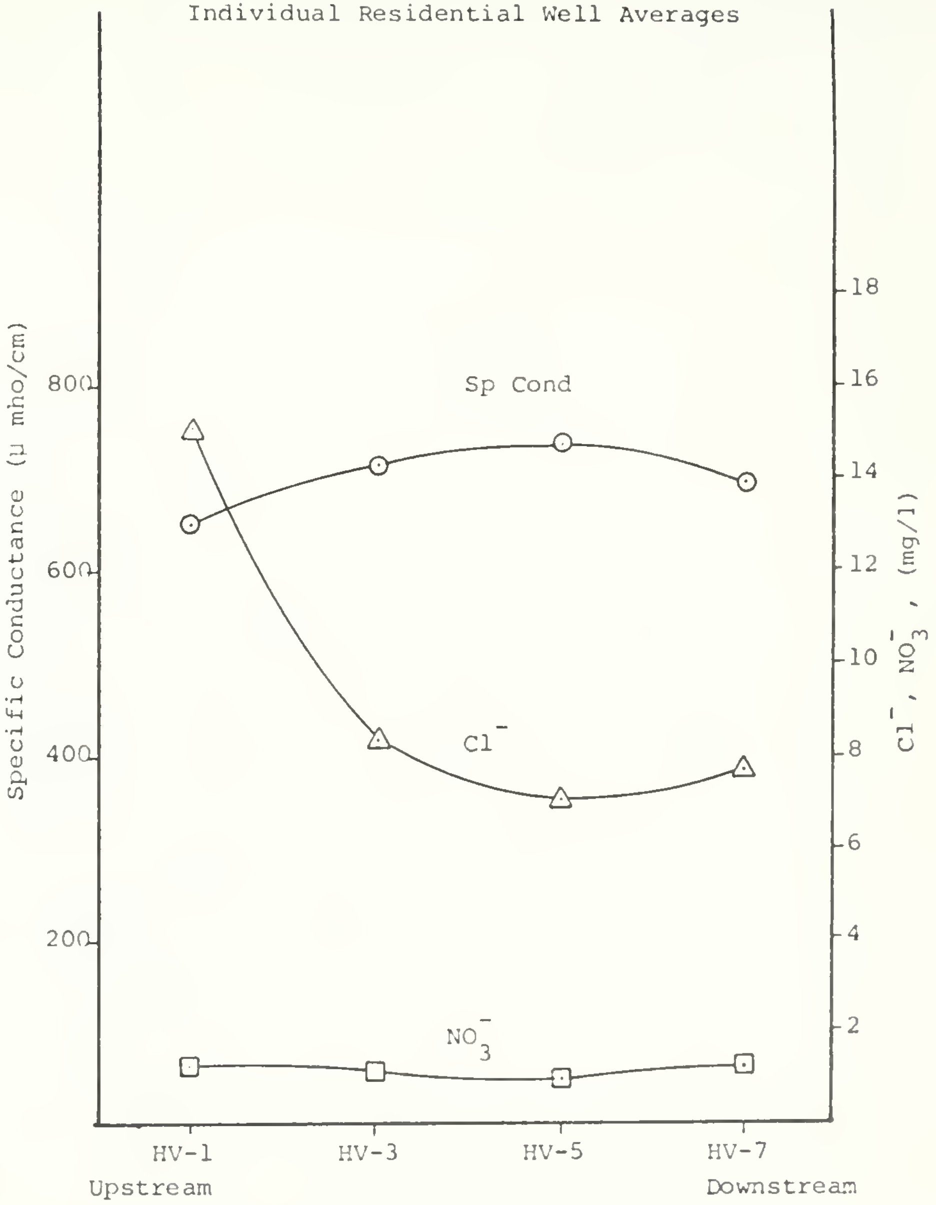
Fig. 8 Livingston
Individual Residential Well Averages

Fig. 9 Sewell



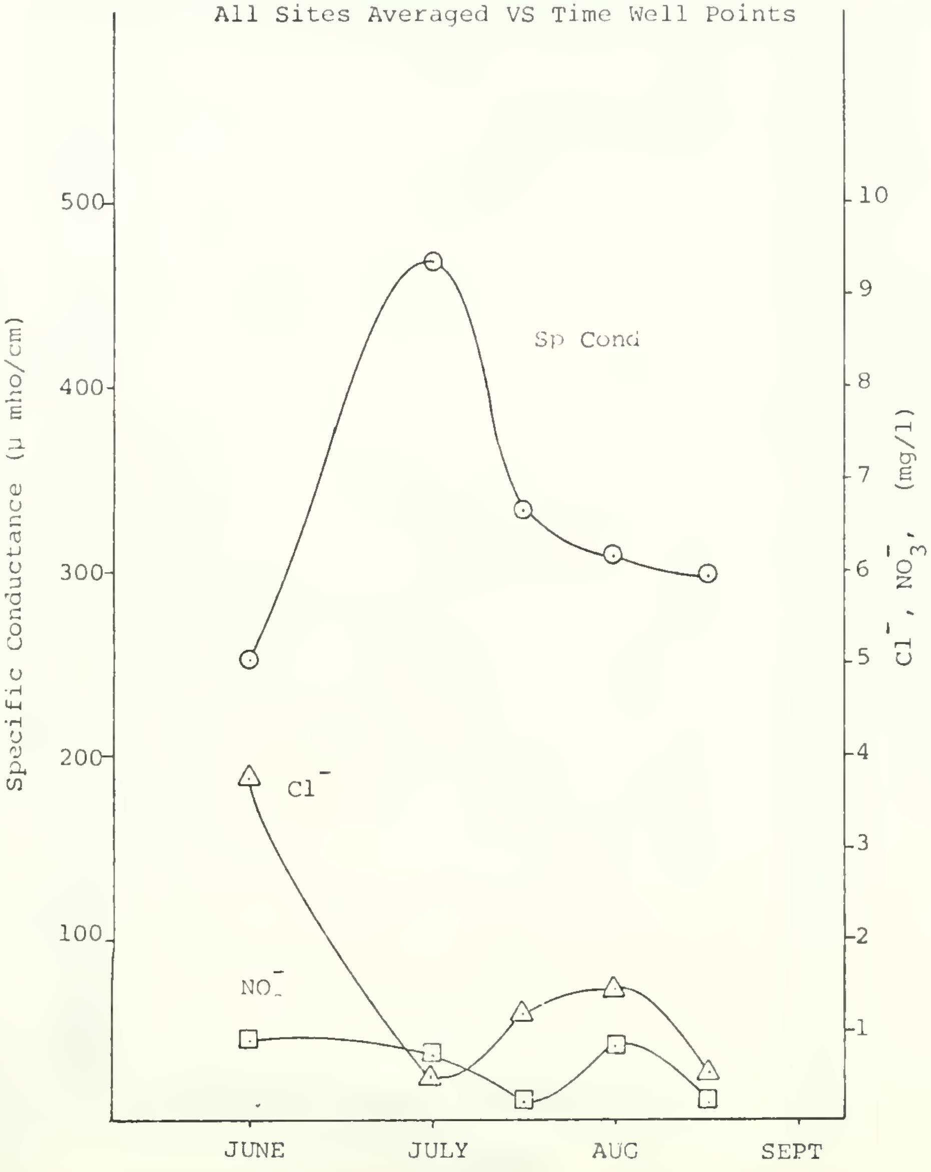
Individual Well Point Averages

Fig. 10 Sewell



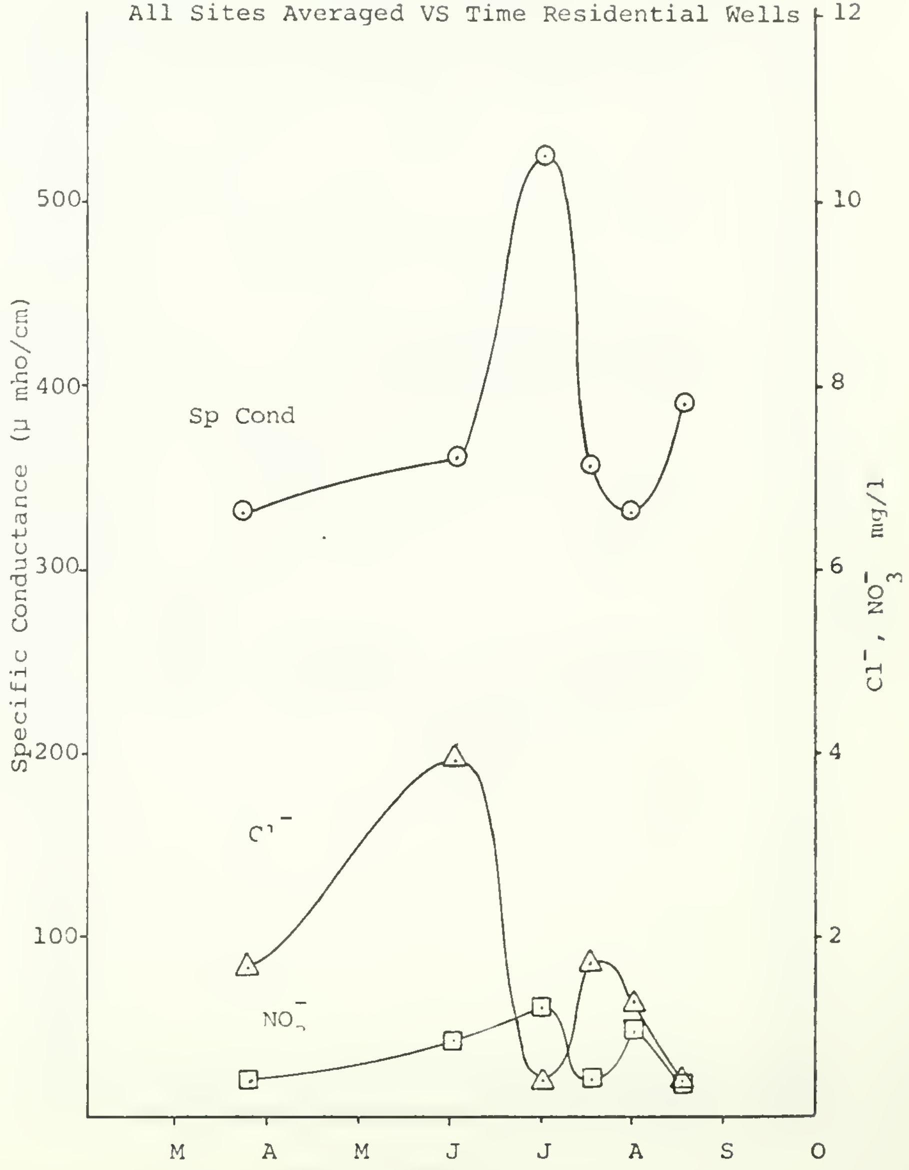
Individual Residential Well Averages

Fig. 11 Entropy Acres



All Sites Averaged VS Time Well Points

Fig. 12 Entropy Acres



All Sites Averaged VS Time Residential Wells

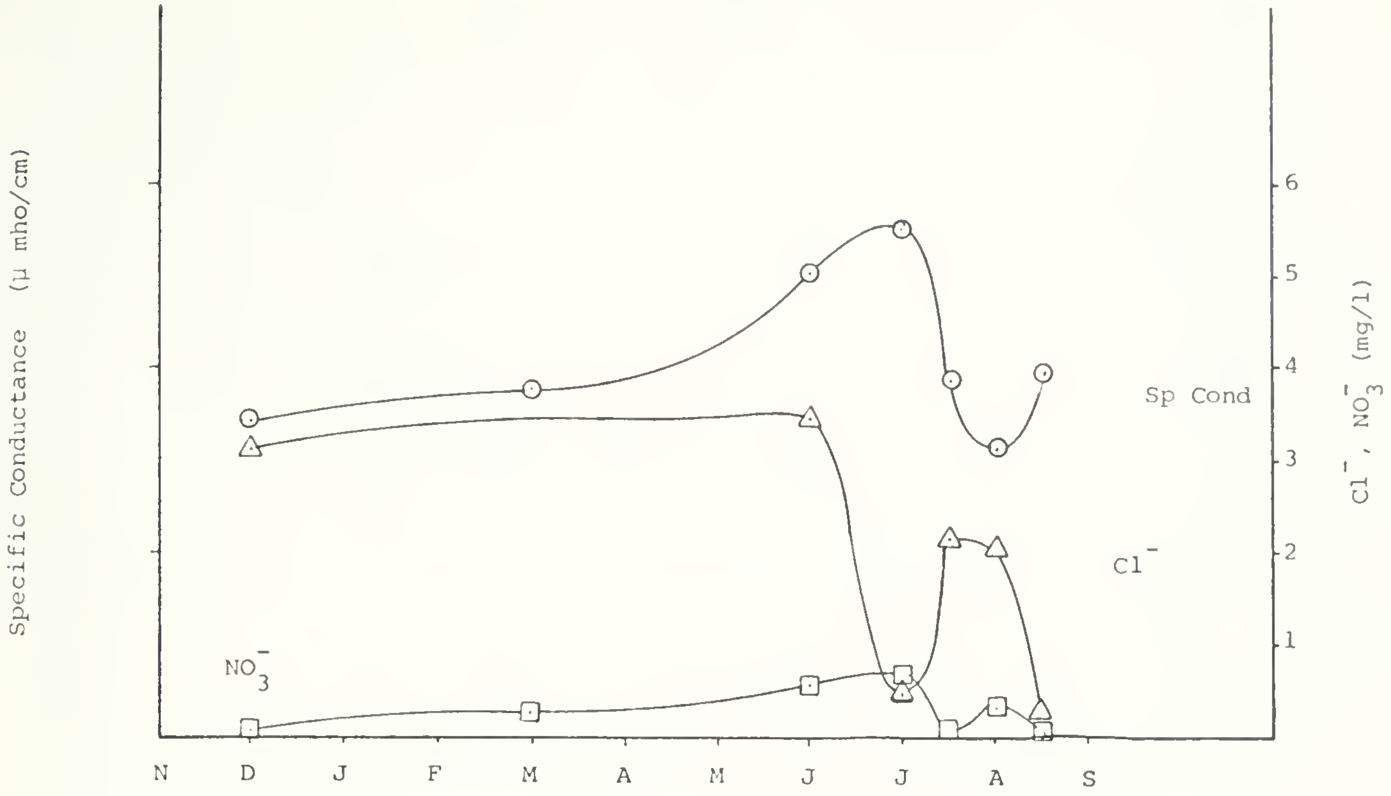


Fig. 13 Hawker Lane

All Sites Averaged VS Time, Well Points

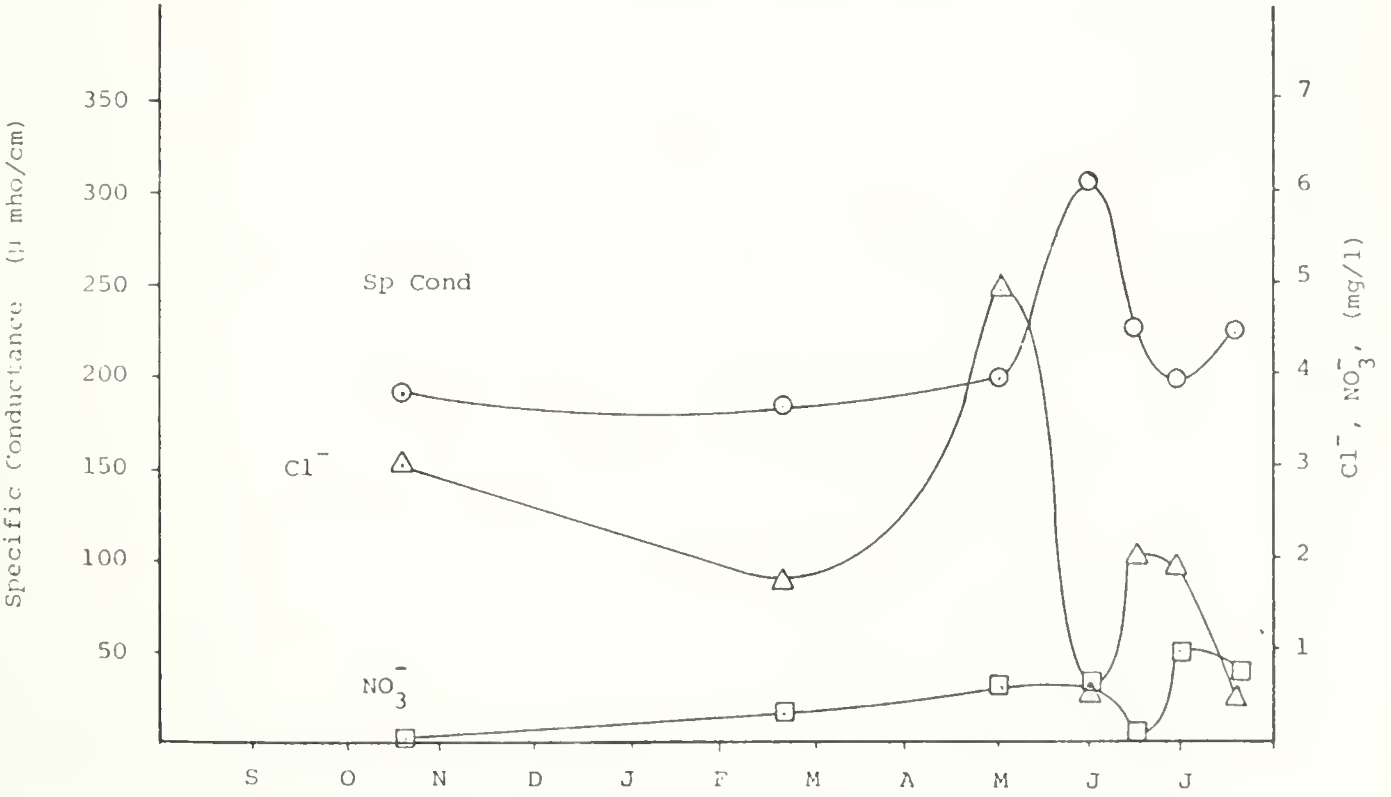


Fig. 14 Hawker Lane

All Sites Averaged VS Time, Residential Wells

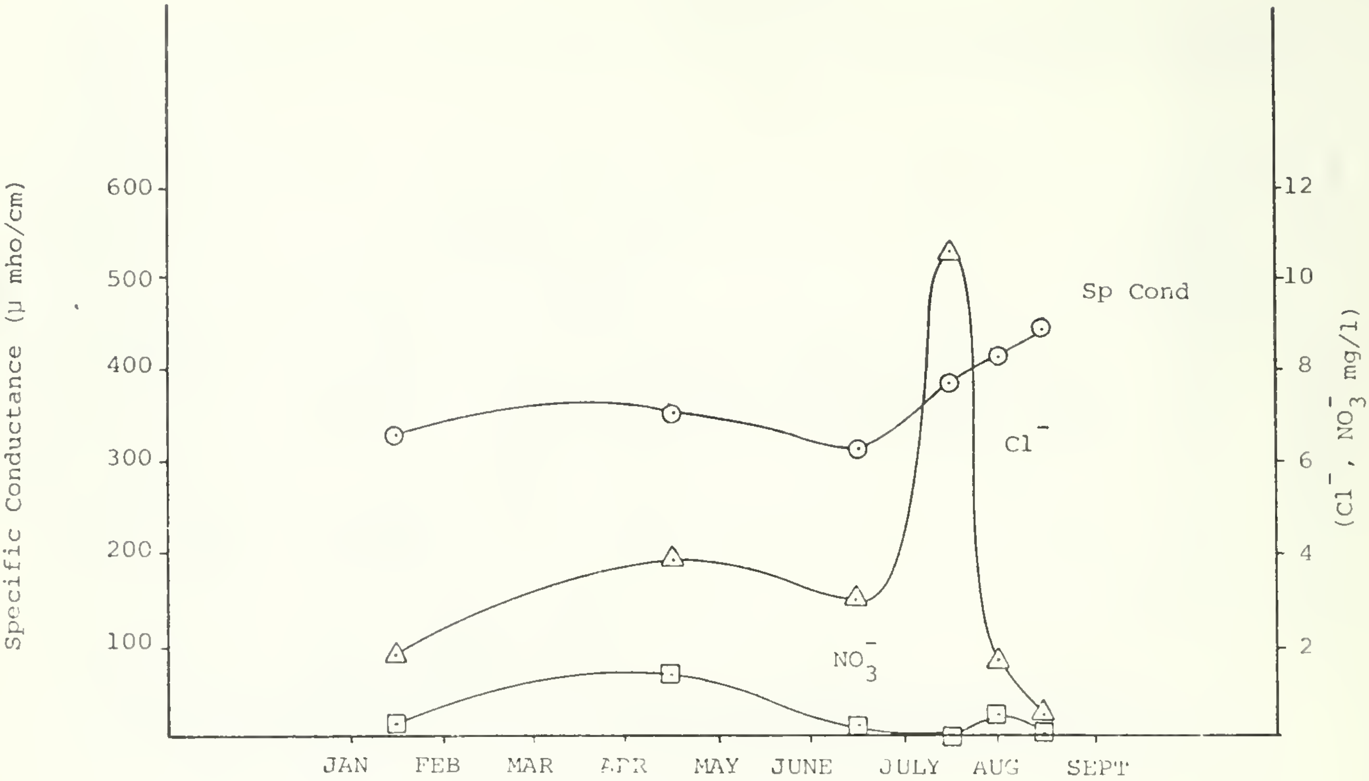


Fig. 15 Lambkins

All Sites Averaged VS Time, Well Points

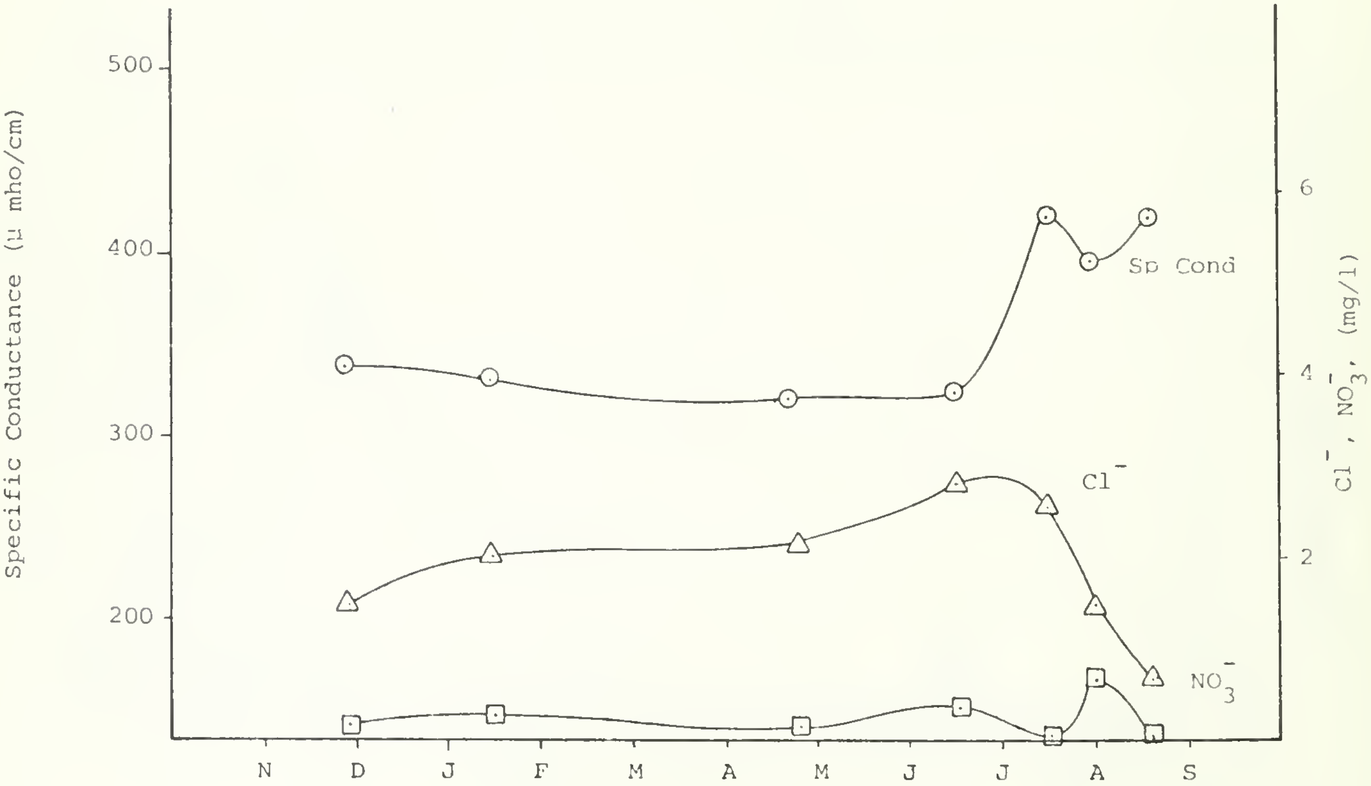
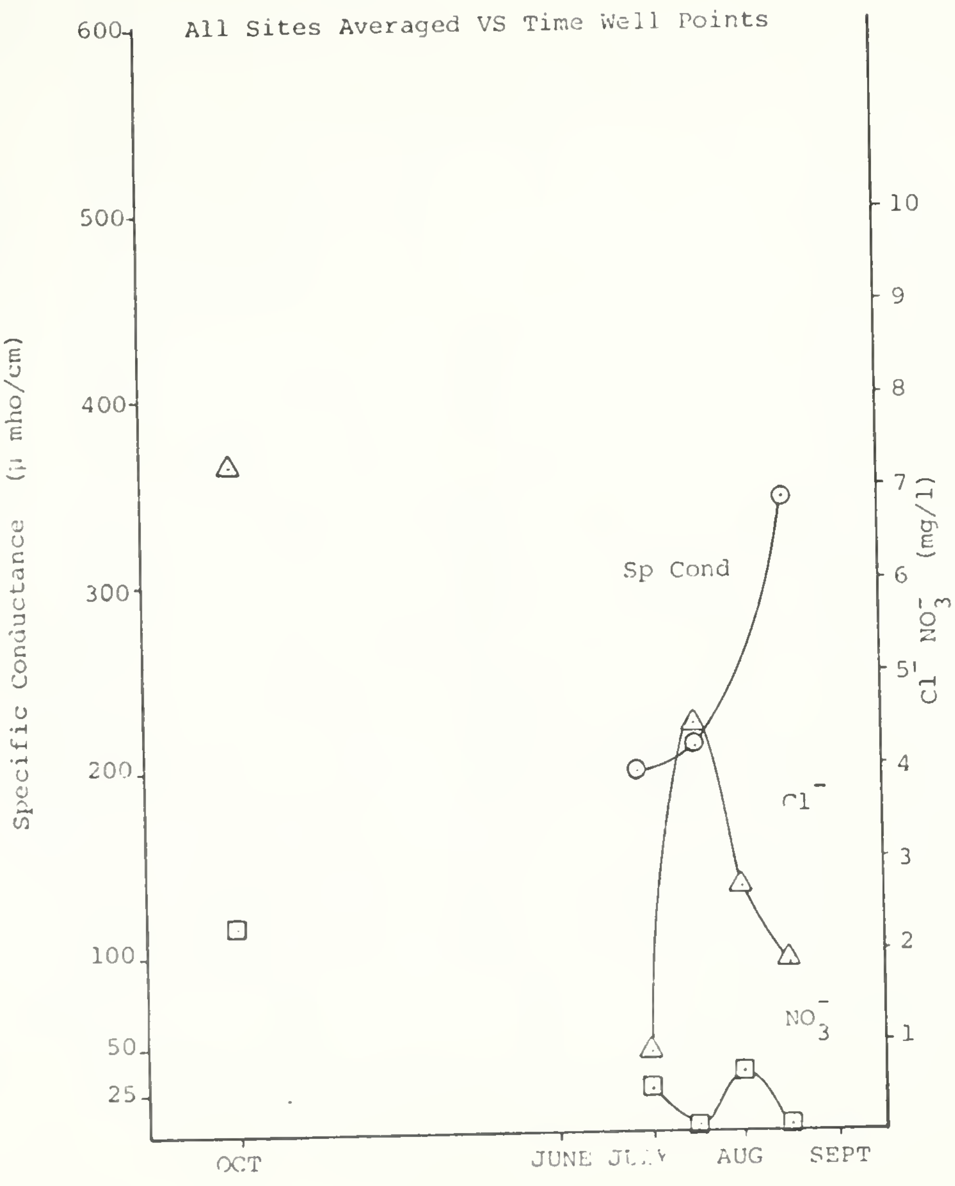


Fig. 16 Lambkins

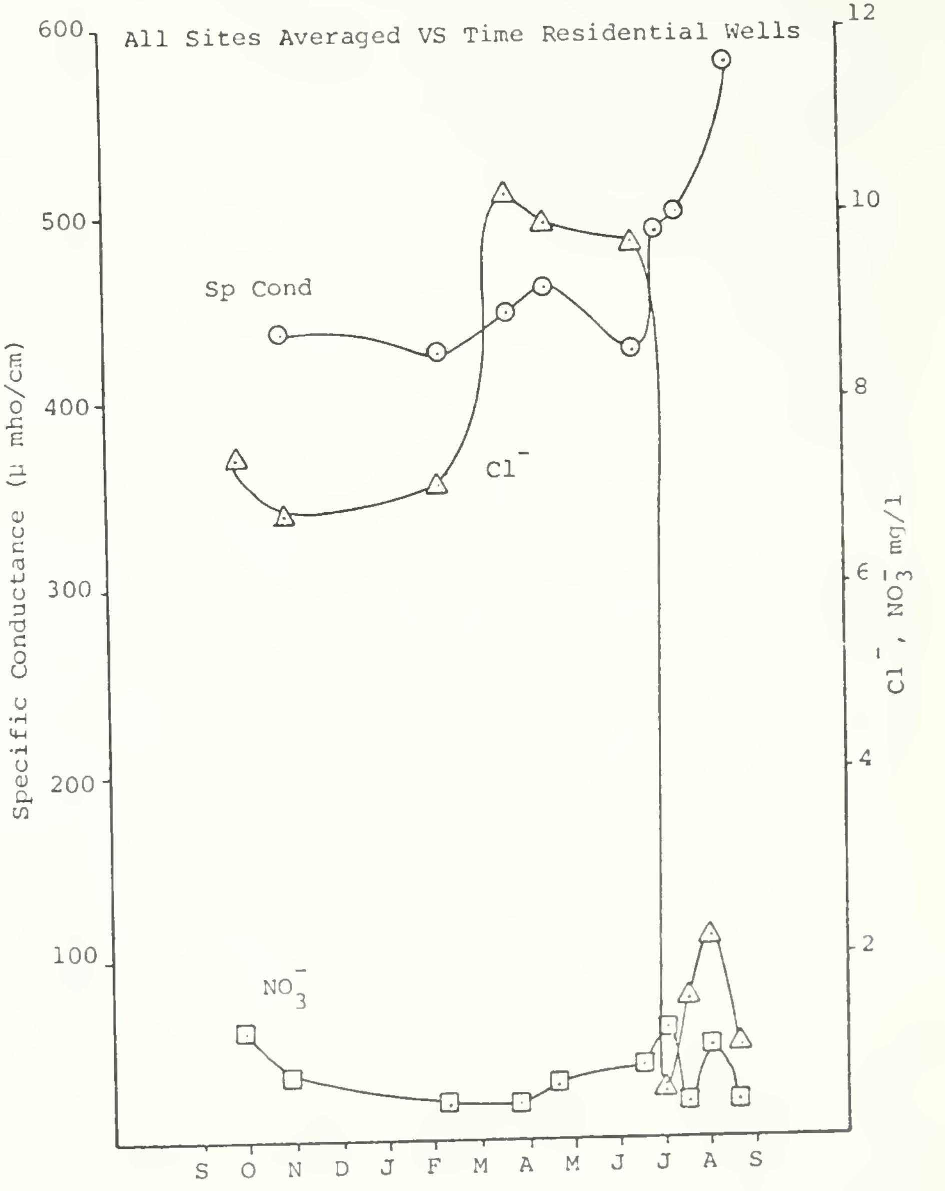
All Sites Averaged VS Time, Residential Wells

Fig. 17 Livingston



All Sites Averaged VS Time Well Points

Fig. 18 Livingston



All Sites Averaged VS Time Residential Wells

Fig. 19 Sewell

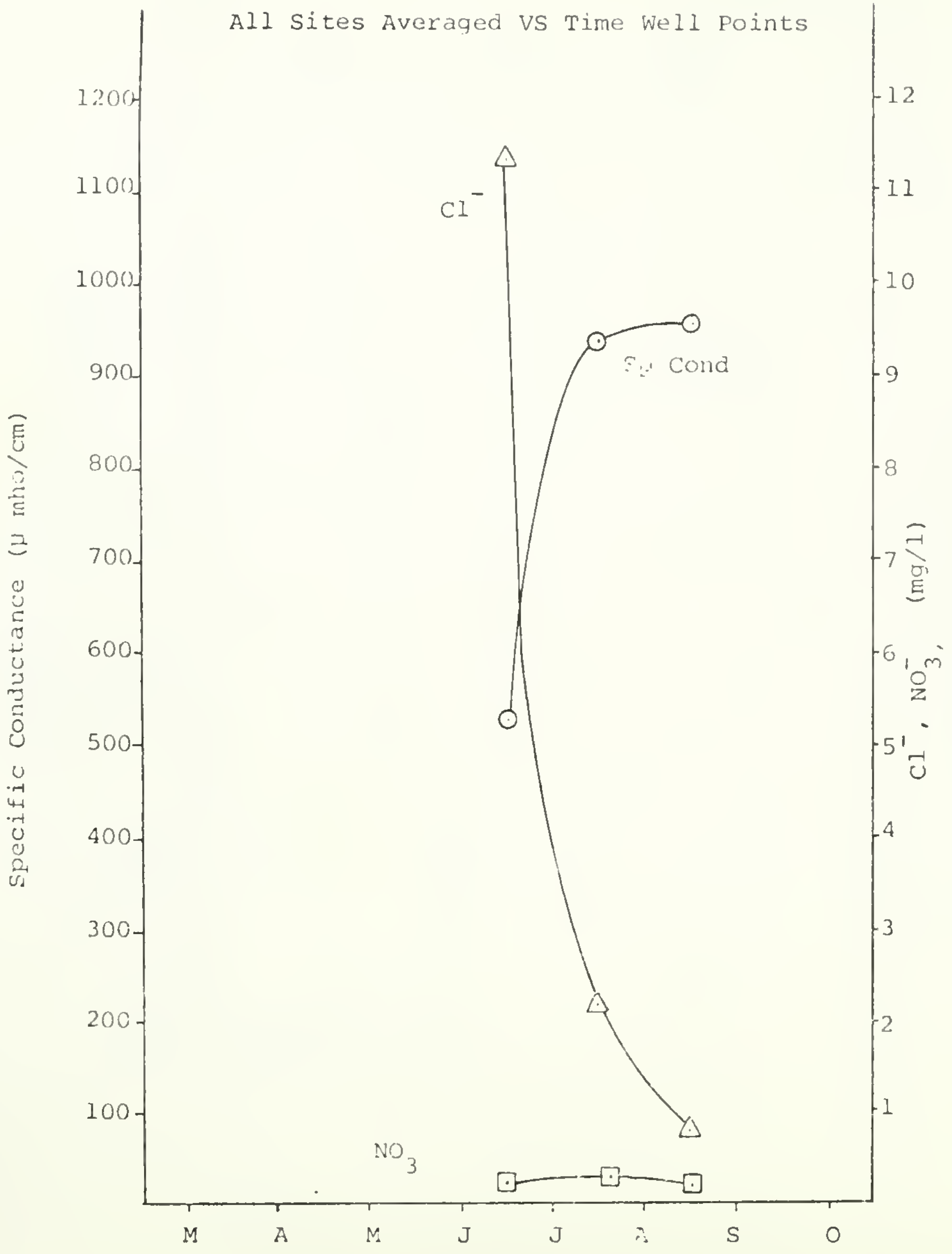
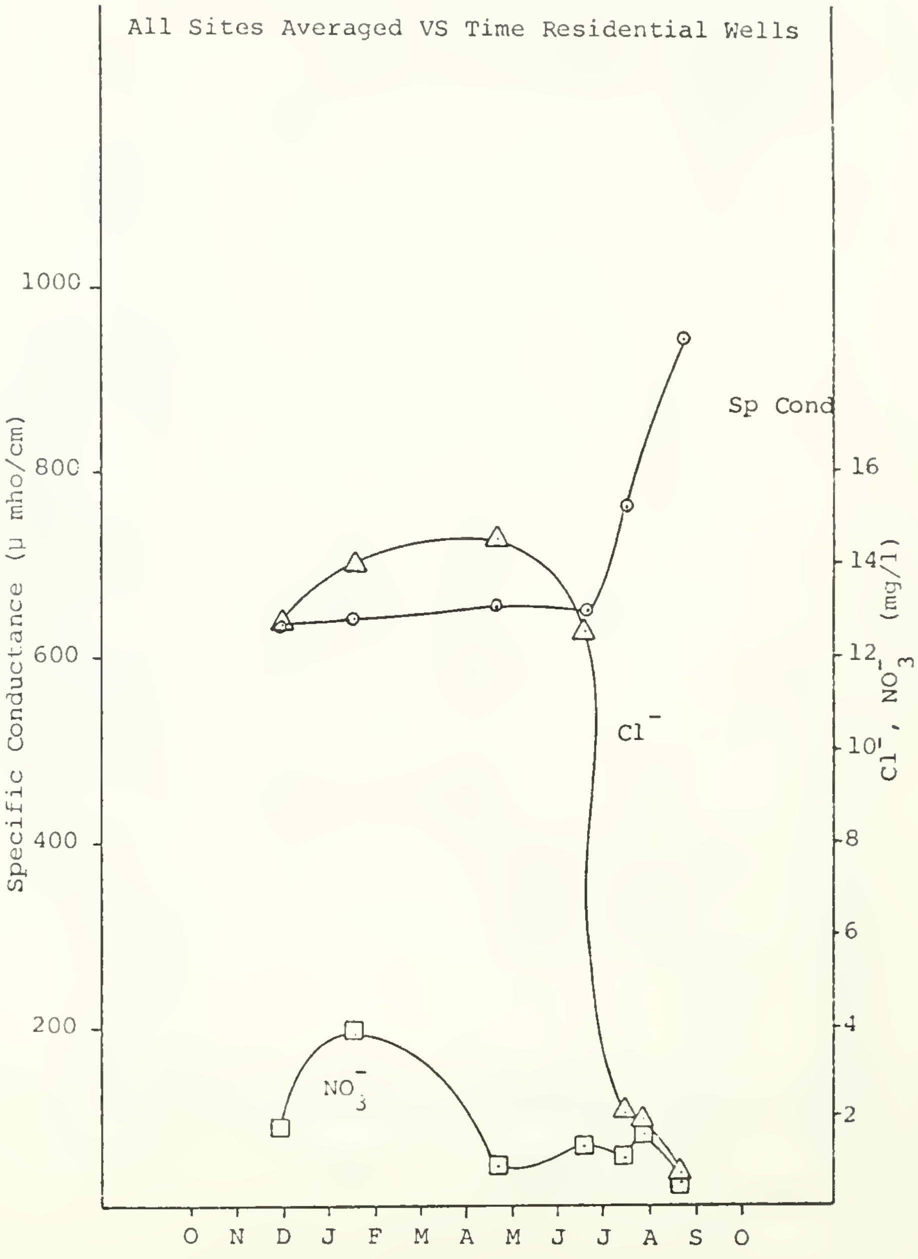

All Sites Averaged VS Time Well Points

Fig. 20 Sewell


All Sites Averaged VS Time Residential Wells

Lambkins (Figure) and Sewell (Figure) showed some increases in conductivity and chlorides. However, the increases were not significant. Other Figures show no distinct patterns of changes.

A correlation analysis using the T Distribution Test was performed to compare the upstream-downstream average site data. The water supply wells in Sewell Subdivision showed a statistically significant change (decrease!) in chlorides at the 93% confidence interval between well no. HV-1 and HV-7. No other sets of data showed statistically significant differences, using the T Distribution Test.

As evidenced by the data in Table 3, the quality of the shallow aquifers sampled via the well points was quite good, as was water sampled from greater depth via the water supply wells.

The above information should be intrepreted and used with extreme care. There are many factors which could influence changes, or lack of changes, in groundwater quality which could not be included in this project. Perhaps of prime importance is the dilution factor of the wastewater by the groundwater. Although groundwater flow could not be measured in this study, it is believed that the volume of water moving beneath all of the sites investigated was substantial. All subdivisions studied were on alluvial soils with shallow layers of topsoil overlaying sand, gravel and cobbles. This porous material, coupled with steep groundwater gradients undoubtedly produced rapid flow rates which would sweep away and dilute material draining downward to the groundwater table.

Additionally, none of the subdivisions studied were fully developed. At approximately 5.4 persons per acre, Sewell Subdivision near Helena has the greatest population density. However, many of the residents of this area are older couples and water use is probably considerably less than could be expected from residences accepted by families with children at home. Thus, the quantity of waste water generated by residents of the subdivisions studied was much less than should be expected from fully developed subdivisions.

The voluminous groundwater flows coupled with minimal waste water flows would produce a large dilution factor which would result in little noticeable change in groundwater quality resulting from subsurface waste discharge.

EVALUATION OF MONTANA CODES

Requirements for the installation of soil absorption systems in Montana are outlined in the Montana Subdivision Regulations (ARM 16-2.14 (10)-S14340). Among other factors, this code addresses depth to ground water and impervious strata, horizontal distances to wells, streams and structures, and hydraulic conductivity of soils. A summary of these criteria is presented in Table 7 below. Additionally, minimum lot sizes of one acre are imposed when both soil adsorption system and water supply well serving the residence are located on the same lot. Where only one of the above are located on the lot, a minimum size of 20,000 square feet is imposed.

TABLE 4

Summary of Design Criteria For Soil Adsorption Systems in Montana

Distance from Soil Adsorption System To:	Vertical (ft)	Horizontal (ft)	Remarks
Groundwater	6		From natural soil surface
	4		From bottom of trench
Bed rock or other impervious strata	6		From natural soil surface
	4		From bottom of trench
Surface stream		100	To high water mark of 100 year flood
Well		100	
Property line		10	
Foundation wall		5	
Water pipe		10	

These regulations were adopted in 1973. Prior to this time, less stringent regulations were in effect. These older regulations required only one-half acre lots for both well and soil adsorption systems and only 2 feet from the bottom of the trench to ground water. Prior to 1961, there were no regulations for installation of soil adsorption systems.

The subdivisions chosen for study in this project were platted prior to the 1973 regulations. They did not meet the more stringent regulations with respect to ground water depth, and in some cases, lot sizes. Selection by the State Department of Health and Environmental Sciences included "worst case" analysis to determine if Montana is currently experiencing ground water quality problems due to soil adsorption systems in sub-standard and marginal areas. The study was not designed to evaluate the effectiveness of existing regulations with respect to soil adsorption systems.

The temptation is to conclude that since "worst case" subdivisions with sub-standard installations are causing no detectable problem, existing regulations are more stringent than necessary. The writers of this report caution against this conclusion. It was previously pointed out that none of the subdivisions studied were fully developed. Thus no information was obtained in this study from a large area at full development. Other studies (Miller 1972, Schmidt, 1972, Quan et al, 1974, Morrill, 1973) previously discussed reported problems in such areas.

Another factor which was previously discussed was the dilution capacity resulting from voluminous ground water flows. Although similar conditions exists in much of Western Montana, other parts of the state may have much less gound water available for diluting waste.

In summary, the writers of this report feel that the existing regulations should be adequate to protect ground water quality under most circumstances encountered in Montana. In some instances (high ground water with low dilution capacities and large fully-developed subdivisions) ground water problems are conceivable. In other areas (small subdivisions, high dilution capacity) perhaps greater densities than presently allowed could be permitted. However, since future conditions with respect to size of area underdevelopment, and perhaps even ground water flows due to changing land use and irrigation practice, cannot always be predicted, the writers of this report recommend no changes in the minimum criteria for the installation of soil adsorption systems.

ALTERNATIVE SOLUTIONS

Montana codes do not include regulations for individual wastewater disposal systems other than ST-SAS. There are, however, provisions for conditional approval of alternative systems if they are termed "experimental". Use of alternative systems is discouraged, since proof of the performance of these "experimental" systems is the responsibility of the owner.

Enforcement policies that recognize ST-SAS as the only method of wastewater disposal for single family dwellings not serviced by collective sewage treatment facilities may interfere with optimum land use planning. For example, limiting on-site wastewater disposal to ST-SAS imposes restraints on the use of some locations for home building. The planning process whereby different areas are approved or disapproved for residential development becomes less flexible. Areas otherwise suitable for housing may be disapproved due to groundwater and soil limitations if the homes are to use ST-SAS. As a result, many prime agricultural areas become the first choice for residential development. In view of these facts, closer attention should be given to the growing body of knowledge aimed at providing alternatives for conventional ST-SMS.

An additional consideration concerning ISDS of all kind should be flow reduction through water conservation practices. Reduced flow improves the economics of ISDS and decreases the likelihood of water pollution from both conventional and alternative systems.

ALTERNATIVE INDIVIDUAL SEWAGE DISPOSAL SYSTEMS

Alternative ISDS technology has been increasing rapidly during the past few years. Design parameters and schemes are available that help to minimize the potential risk of system failure if an alternative ISDS is employed for wastewater disposal. Information that will help officials to select the various forms of alternative waste disposal most suited for a given area may be obtained from authoritative sources.

As noted earlier alternative waste disposal technology enables the development of marginal areas for residential use without introducing unacceptable health risks. Additionally, failing ST-SAS installations serving existing dwellings may be corrected by use of one of the alternative waste disposal systems.

Not all alternative systems are suitable for all locations. Care must be taken to select only those systems which will function within the limitations imposed by the design and the location in question. Several ISDS commonly proposed as alternatives to ST-SAS were examined and are discussed below. The systems are divided into two groups, those which appear suitable for use in Montana and those which the authors felt were not suitable for use in Montana. Criteria for this dichotomy include: (1) ability to function under climatitic conditions encountered in Montana, (2) the body of data available to confirm general effectiveness, (3) the level of operational and maintenance expertise required, (4) capital and O & M costs, and (5) whether or not an ultimate surface discharge is involved. Alternative ISDS were recommended only if (1) sufficient data indicated that they will work in Montana, (2) no surface discharge is involved, (3) routine operation is within the expertise of the average person, and (4) if costs are competition with ST-SAS.

Systems With Possible Application in Montana

Mound Systems

A mound system, an engineered artificial soil bed constructed of selected soil, sand, and gravel, is one method of disposing of pretreated wastewater on sites unsuited for subsurface soil disposal. The sand fill provides additional soil material for treating wastes before they reach groundwater and increases the surface area interface between the fill and natural soil at sites with slowly permeable soils.

A mound system consists of a septic tank, pump or siphon chamber and a mound. The mound itself consists of a sand fill elevated above the original soil surface, an adsorption area, and a clay or silt cap. Effluent is pumped up to the adsorption trenches or adsorption bed located in the mound's interior in measured doses and is distributed under hydrostatic pressure evenly throughout the distribution lines.

Within limits, mound systems may be utilized where high ground water, creviced or channeled bedrock, or soils with fast percolation rates prevent adequate treatment of the effluent prior to its discharge into the ground water.

In these designs, the fill material must serve two purposes: (1) provide sufficient depth for treatment to occur in the fill material and (2) distribute the effluent evenly onto the natural ground surface so that percolation rates through the natural soil will be minimal.

Mounds may also be designed for use in areas with slowly permeable soils. In such cases, the purpose of the mound is to increase the area available for infiltration and percolation. The porous fill material spreads the flow over the entire surface of the natural ground area to be used, thereby increasing area available for infiltration. This increased area compensates for the slow infiltration rate. Care must be taken that horizontal flow from underneath the mound ("daylighting") does not occur. (Goldstein and Maberg, 1973)

Limitations for mound use are imposed by topography, percolation rates, depth to ground water and depth to bedrock. Set-back distances and siting conditions are requirements that must be met prior to building a mound. Table 5 summarizes the soil and site factors that restrict mound construction.

TABLE 5

Soil and Site Factors that Restrict Mound Construction
(Converse, et al, 1977)

	Restricting Factors					
Soil Group	Depth to Seasonal High Water Table	Depth to Impermeable Soil Layer or Rock Strata	Depth to Pervious Bedrock	Depth to 50% by Volume Rock Fragments	Percolation Rate (min/inch)	Slope %
Slowly Permeable Soils	20 inches	20-60 inches	20 inches	20 inches	60-120	6
Permeable soils with shallow pervious bedrock	--	--	20 inches	20 inches	30-60	6
Permeable soils with high water tables	20 inches	20-60 inches	--	20 inches	30-60	6

Sufficient lateral movement is required to reduce the amount of effluent under the mound. Soil percolation and terrain slope influence lateral movement. Slowly permeable topsoils require greater depths to the impermeable layer, as do sloping sites that restrict effluent movement in one direction. Frost penetration will reduce the effective area for lateral movement. Increased mound area is needed to compensate for large amounts of rock fragments or poor soil texture, since these factors may decrease the effectiveness of waste water renovation.

Siting has to be considered before establishing a mound. Landscape position, flood plains, drainage ways and depressions are factors associated with siting. Locations that require major site work may be unsuitable for a mound. Extensive surface preparation with heavy equipment could smear, compact, and alter the infiltrative capacities of the site.

Climatological conditions do not appear to effect performance of a properly designed mound system (Otis and Boyle, 1979). Mound systems have worked well under the adverse climatological conditions experienced in North Dakota, where they were first designed (Goldstein and Moberg, 1973).

Design. Several factors must be considered when designing a mound. The adsorption area within the mound is determined, the mound is dimensioned, basal area requirements are found, the distribution network is laid out, and the pumping system planned. A balance must exist between the adsorption area size and desired purification. A rapid liquid movement through the mound will reduce the adsorption area size but will also reduce the mound's renovation efficiency.

Mound dimensions are a function of site conditions, loading and climatic factors. The most critical dimensions are the depth of fill beneath the trench or bed, bed or trench area, spacing between trenches, and downslope distances between the trench or bed edge. Basal area size is dependent upon the natural soil infiltrative capacity.

The distribution system is constructed of a 2-inch manifold and 1-1/4 inch laterals with evenly spaced, 1/4 inch holes. Uniform application of the effluent is required to prevent saturation of the mound in one particular area with a subsequent loss of renovation capacity.

The pumping system is designed to provide a sufficient quantity of effluent for each dose in order to give uniform distribution. Dosing frequency must be balanced with dosage quantity. More frequent dosing with smaller quantities per dose reduces the chances of mound saturation and leads to better purification, but dosing too frequently will cause crusting and poorer distribution.

Since quantity per dose is the most important parameter, liquid level control in the pump chamber is important. The pump must be sized to supply two feet of head at the most distant end of the laterals. Explicit design plans are available from the Small Scale Waste Management Project at the University of Wisconsin.

Treatment Capability and Costs. Mounds have been studied to determine their effectiveness in renovating wastewater. Six mound systems were studied by the University of Wisconsin in the early 1970's. Four of the six mound systems operated satisfactorily (Bouma, et al, 1972). Unsatisfactory mound performance was felt to be due to inadequate design.

A properly designed mound will remove nearly all BOD_5, S.S., and pathogenic bacteria passing through the sandfill prior to reaching the natural soil surface. There is complete nitrification in the sandfill, but no nitrogen removal. Phosphorus is not removed by the sandfill, but viruses are completely removed (Otis and Boyle, 1979).

Capital costs vary from $.75 to $3 per square foot of basal area depending upon distance traveled to obtain suitable fill material (Otis and Boyle, 1979). Mounds for Cedarburg and Grafton, Wisconsin were determined to cost approximately $3,400 to $4,700 per unit (Nall, 1978). These costs may differ in Montana.

Evapotranspiration Systems

There are two types of evapotranspiration (E-T) beds. In one design, soil infiltration and evapotranspiration are combined. Another design is non-discharging, relying solely on evapotranspiration.

The combination system was first proposed by Bernhardt for use in tight soils where high ground water is not a problem. Shallow trenches, approximately 20 inches deep, are employed. Crushed stone, gravel and sand are placed under the pipe, while sand and topsoil cover the pipe. The objective of this system is to enhance evapotranspiration while utilizing the available seepage.

Bernhardt has developed formulas for estimating the required bed size based on the loading rate, wastewater strength, a seepage factor, ventilation coefficients and an evaporation factor. A five-step calculation and design procedure was developed by Bernhardt that included calculations for determining the building area, drainage area, green area, well protection area and seepage area. An example of the design procedure is included in Wastewater Treatment Systems for Rural Communities.

Non-discharging E-T systems may be used in areas with high ground water, tight soils, or both, provided suitable climatic conditions exist.

The system consists of septic tank pre-treatment, and an E-T bed. A continuous PVC liner is placed in the bottom of an excavated bed. A four-inch pipe surrounded by gravel is located in the bottom of the bed, and 18 to 30 inches of carefully selected sand is placed on top. The selected sand must be in the size range to cause capillary rise of water to the surface.

Sizing of the system is based on in-flow rate, precipitation and E-T. Potential evaporation may be established by 1) energy balance techniques, 2) empirical equations based on meteorological data, or 3) measured pan evaporation.

Studies were conducted by the University of Colorado and the University of Wyoming to establish operational parameters and design criteria for non-discharging E-T systems. The University of Colorado found that average year-round evaporation from an E-T bed was approximately forty percent of the pan evaporation rate (Hasfurther, et al, 1977).

The University of Wyoming used empirical equations to establish E-T. E-T is influenced by solar radiation, temperature, elevation, relative humidity, wind speed, soil moisture availability, plant density and surface area. Three methods were tested to determine the most suitable empirical equation for the Laramie area. The Blaney-Criddle method gave the best overall results for evaporation losses from the months of May to October (Hines, et al, 1977).

Conservative engineering practice assumes that E-T minus precipitation on a month-by-month basis yields a reasonable estimate of the amount of wastewater that E-T will remove. Thus, this type of system is only suitable for semi-arid regions where evaporation substantially exceeds precipitation.

Use of plants to increase moisture loss is effective during the summer months. Plants used with an E-T system must have a wide tolerance to soil moisture content. Soil moisture content in an E-T bed will change radically over the summer months.

Estimated E-T costs will vary depending upon location. They may range from $1.00 to $1.25 per square foot for installations in Colorado (Hines, 1977). Researchers in Wyoming estimated the cost to range from $1450 to $3200 per installation for their design. This is compared to a ST-SAS cost of $1300 to $1500.

Cluster Development With ST-SAS

Wastewater from individual septic tanks may be collected and transported by small diameter sewers to a central adsorption site. This system may be utilized for small subdivisions to eliminate the necessity of installing a soil adsorption system on each lot. The individual septic tanks provide primary settling of the effluent and require normal maintenance.

Either interceptors and laterals must be installed on sufficient grade to insure gravity flow or a pressure sewerage system must be used. The area to be utilized for soil adsorption must be capable of handling the combined septic tank flows. Additional review by governmental agencies may be required prior to installating a system employing this concept.

A management system must be established to allocate responsibility of the system among the various homeowners. Although the management arrangement may take many forms, it must be clearly established to address any legal problems arising from this arrangement.

Advantages of this type of system can include increased maintenance of the soil adsorption system and septic tanks by a recognized formal authority, elimination of a soil adsorption system on each lot, relatively low capital costs, and relatively low maintenance requirements. The system is easily adapted to developments where existing homes already have septic tanks and primary treatment is provided onsite.

Systems Not Recommended For Use In Montana

Sand Filters

There are three basic designs of sand filters: buried, intermittent and recirculating. All sand filters act as mechanical filtration units. Sand particle size varies among the different types of filtration designs but particles are generally 1.0 - 1.5 mm for buried sand filters, .3 to .6 mm for intermittent sand filters and 1.0 to 1.5 mm for recirculating sand filters. Uniformity coefficient for the sand utilized is approximately 3.5.

Sand filter systems generally consist of a primary treatment system (usually a septic tank), a dosing system for applying the settled sewage, and the sand filter. An additional recirculation tank is required for recirculating systems.

The settled sewage is distributed over the filter surface either by a splash pad, in the case of intermittent systems, or by distribution lines in the case of buried or recirculating systems. The applied effluent then filters down through the sand (24 inches deep for intermittent sand filters and 30 inches deep for recirculating filters) and is collected in underdrains surrounded by gravel.

Loading rates are about 3 gallons per square foot per day for both intermittent and recirculating filter units. Effluent quality is generally acceptable, with BOD and S.S. having values less than 10 mg/L.

Operation and maintenance consists of routinely checking the dosing system and periodically raking the surface area. Once a year physical break-up of the crusted filter surface and a 30 to 60 day rest period is required to restore infiltrative capacity. This requirement means two filters must be available to allow for resting of the filter system. Approximately once a year the upper 2-4 inches of sand must be scraped off and discarded to reduce clogging problems.

An intermittent sand filter system was evaluated by the Small Scale Waste Management Project at the University of Wisconsin from 1973 to 1976. The filter system was enclosed in a partially buried concrete block basins and covered by an insulated cover. Loading rates normally fell within 2-10 gals/ft^2/day. The effluent was dosed by a pump at 40 gallons per dose with as many as 13 doses per day.

Although effluent quality was generally good, chlorination was required to control microorganisms. The study determined that a design loading rate of 5 gallons per day per sq. ft. should be used with filter runs from 6 months to one year.

In addition to periodic maintenance, sand filtration requires an ultimate point of discharge. Due to these and other disadvantages, sand filters are not recommended for use in Montana at this time.

Aerobic Aeration Units

Most aerobic treatment units employ batch process methods in which aeration and settling are performed in the same chamber. The aeration chamber provides an environment suitable for maintaining the growth of microorganisms which are required to biologically stabilize the wastewater. The size of the units generally range from 70 to 175 ft^3 and include an aeration compartment and settling chamber and sometimes a pretreatment compartment (Hietzler et al., 1977).

Flow regimes vary among the different designs. The simplest and most common flow configuration discharges effluent at a rate that is in direct response to inflow. This flow design is subject to problems induced by influent surges.

Aeration is normally accomplished by forced air systems or mechanical devices. Mechanical aeration devices may include surface mixers or subsurface mixers with draft tubes.

Home aerobic treatment units may employ either suspended growth or attached growth reactors. The suspended growth design is more common than the attached growth. Attached growth systems have the advantage of being less susceptible to upset since hydraulic surges are less likely to wash solids out. Mixing, solids separation processes, and sludge return is eliminated by the use of attached growth media. However, head losses and temperature effects are major disadvantages of attached growth systems.

Several surveys have been made pertaining to aerobic system use and performance. Voell and Vance (1974) studied 151 units over a two year period and found that 10% of the units experienced control problems, 75% of the dry feed chlorinators were empty and 57% lost solids into the effluent. Tipton, in 1975, surveyed 50 units and found that most could not meet a state discharge standard of 50 mg/L B.O.D.

The effluent quality of aerobic units was characterized as being low in organic and solid material but was also highly variable due to periodic upsets caused by biological and mechanical problems (Hutzler, et al, 1977). The average BOD_5 reported by Hutzler was approximately 37 mg/L.

Mechanical problems were noted with aerobic treatment units. The difficulties most often associated with aerobic treatment systems were caused by the blowers (Hutzler, et al, 1977). The blowers were found to have an average life span of only two years, indicating that most airblowers presently being used are unsatisfactory.

Chlorinators also contributed to the problems associated with aerobic units (Hutzler, et al, 1977). The units became plugged or lost their prime.

Routine maintenance includes parts lubrication, clarifier cleaning, equipment checks and solids removal. Surface discharge units should be sampled periodically to insure proper system performance. A homeowner must be able to detect when his system is malfunctioning and also be able to determine when the installed system is operating correctly.

It is generally agreed that most homeowners have neither the expertise nor incentive to maintain their own treatment units. Regulatory agencies discourage the use of aerobic units due to homeowner misuse, improper maintenance, and lack of data (Hutzler, 1977). The problem of homeowner misuse and lack of understanding becomes greater as the unit's design becomes more complicated.

Total Recycling Systems

Technology is being developed that would allow non-consumptive reuse of wastewater after treatment. A recycling system would eliminate the need of a conventional septic system and allow development of marginal land. Water conservation would be greatly advanced by the use of recycling technology.

The components of a typical recycling system include biological digestion, ultrafiltration, organic adsorption, demineralization and ultra-violet sterilization units and a storage tank for treated water. The wastewater generated by a household is first subjected to biological digestion and settling. Particulate removal is accomplished by ultrafiltration, after which the wastewater is tested for clarity by a nephelometer. The wastewater is then passed through media which induces carbon adsorption and demineralization. A conductivity sensor determines the effectiveness of carbon adsorption and demineralization. Sterilization of the treated water is the last step in the treatment cycle. The treated water is then pumped to the holding tank.

One system design uses a microprocessor to monitor critical points in the wastewater treatment process. Failure of one or all of the compenents in the treatment scheme would automatically turn the system off.

Recycling systems have not been subjected to performance analysis over long periods of time and long-term treatment effectiveness has yet to be determined. Until definitive performance data is available, the system's ability to operated properly remains in doubt. Additionally, recycle systems require active operation on the part of the homeowner, the implications of which have been previously discussed.

FLOW REDUCTION TECHNIQUES

Whether ST-SAS or an alternative ISDS is used, water conservation practices may be used to reduce the flow rates that must be accomodated by an individual sewage disposal system. A variety of plumbing devices and appliances are available that will effectively reduce projected wastewater flows (Otis and Boyle, 1979). Water-saving devices may be classified as active or passive and reliable or unreliable. If the proposed device requires an active role by the consumer, the projected wastewater flow reduction is usually unreliable.

Non-Functional Water Use

Elimination of non-functional water use will reduce wastewater flows. Wasteful water habits, excessive water pressure (greater than 40 psi), and inadequate plumbing and appliance maintenance (dripping faucets) all contribute to non-functional water use. Unfortunately, the elimination of non-functional water use requires the active participation of the consumer, thereby reducing the likelihood of its effectiveness.

Water-Saving Devices

Toilet flushing, bathing, and clothes washing account for 70% of the typical household interior water use. One flush of a conventional toilet uses 4.3 gallons; a typical bath or shower consumes 24.5 gallons of water or 20% of an individual's total daily contribution. A clothes washer contributes 37 gallons per use or 22% of the total daily flow (Siegrist, et al, 1977). Devices available for reducing flows from these sources vary in price and effectiveness. Available devices for flow reduction are summarized in Table 6 (Siegrist, et al, 1977 and Otis and Boyle, 1979).

Most of the devices delineated in Table 6 are self explanatory: however, some items pose special problems. Very low volume toilets generally use water and compressed air, thereby requiring the installation of an air supply source. Composting toilets have experienced problems with accumulation of fluids, fly infestation, odors and residue infectiveness, generally due to improper care. Incineration toilets have associated problems with noise and odor, inability to handle more than a few successive users, potential fire hazard if installed improperly and high operating costs.

TABLE 6

Water Saving Fixtures and Appliances

Generic Type	Flow Reduction+	% Reduction of Total Daily Flow	Active/ Passive*	Cost Range*** $
Toilet Devices				
Toilet tank inserts	15-45	3.9-15.4	P	2-6
Dual flush systems	15-30	1.9-12.3	A	5-15
Reduced or Non-Water Carriage Toilets				
Shallow trap		6.8	P	10-15
Very low volume toilets		31.0	P	400 + O&M
Non-Water carriage toilets		35	P	
Incineration				700 + O&M
Composting				800 - 1700
Closed loop				4000 - 5000
Bathroom Devices				
Shower flow controls	10-40	0-10	A	5
Reduced flow shower fixture	40-80	0-10	A	1 - 10
Air assisted shower fixture	60-90		A	275 - 300
Low volume bathtubs	0-30		P	0
Sink faucet restrictions	30-90		P	1 - 10
Laundry				
Suds saver	10-30	4.8-6.8**	A	15 - 30
Multi-level cycle control	10-40		A	50 - 75
Kitchen				
Multicycle control dishwasher	10-40		A	50 - 9
Garbage disposal elimination	95		P	---
Sink faucet restrictors	30-90		A	1 - 10

+ Per cent reduction in flow rate when flowing wide open, or in volume per use

* User role in achieving reduction

** Dependent upon number of times wash water recyled

*** Capital cost in excess of conventional

TABLE AVERAGE PARAMETER VALUES

Site	Specific Conductance micromhos/cm	Fecal Coliforms per 100 mls	Cl mg/l	NO_3^--N mg/l	NH_3^--N mg/l	COD mg/l	PO_4-P mg/l	Groundwater Depth ft.
				Parameter				
				Hawker Lane Estate				
HM-1P	186	<1.14	1.8	.3	0.02	8.0	0.04	4.5
HM-2W	244	<1	2.3	.4	<0.01	.6	0.13	NA
HM-3P	178	<1	2.4	.4	0.03	11.4	0.05	5.4
HM-4W	181	<1	1.9	.3	<0.01	2.6	0.07	NA
HM-5W	203	<1	2.0	.5	1.35	4.7	0.15	NA
HM-6W	254	<1	1.8	.3	0.01	.6	0.02	NA
HM-7P	248	<1	2.0	.2	0.03	4.1	0.10	2.8
				Entropy Acres				
HD-1P	416	<1	1.7	.7	2.02	7.0	0.16	3.8
HD-2W	389	<1	1.6	.9	<0.01	1.7	0.08	NA
HD-3P	299	<1	1.4	.6	0.16	9.1	0.09	6.6
HD-4W	381	<1	1.8	.7	0.02	0	0.02	NA
HD-5P	295	<1	1.4	.6	0.02	8.7	0.06	6.2
HD-6W	373	<1	1.6	.6	1.68	1.4	0.05	NA
HD-7P	313	<1	1.4	.6	0.01	6.1	0.09	7.0
HD-8W	383	<1	1.3	.6	0.02	0	0.02	NA
				Five Acre Tracts				
LV-1W	632	<1	4.7	.9	0.01	13.5	0.08	NA
LV-2W	341	<1	3.2	1.3	0.31	15.4	0.06	NA
LV-3W	544	<1	8.1	.7	0.01	15.0	0.05	NA
LV-4W	416	<1	5.6	.5	0.05	16.4	0.08	NA
LV-5W	491	<1.17	5.9	.5	<0.01	12.1	0.06	NA
LV-6P	418	<2	1.1	.5	<0.01	13.4	0.06	5.4
LV-7P	269	<1	2.5	.4	0.01	11.6	0.06	2.8
LV-8P	NS							
LV-9W	NS							
LV-10W	413	<1	3.4	.7	<0.01	21.3	0.05	NA
				Lambkin Addition				
LN-1P	350	<1.5	2.5	.3	0.01	17.9	0.03	9.0
LN-2P	366	<1	2.0	1.1	<0.01	13.6	0.06	2.9
LN-3W	357	<1	1.9	.2	<0.01	8.3	0.08	NA
LN-4W	332	<1	2.2	.3	<0.01	0	0.06	NA
LN-5P	364	<1	2.5	.3	0.02	9.6	0.01	5.2
LN-6W	360	<1	1.9	.2	<0.01	6.0	<0.01	NA
LN-7P	390	<1	2.3	.3	<0.01	18.5	0.12	3.1
LN-8W	363	<1	1.9	.3	<0.01	9.1	0.03	NA
LN-9P	421	<1	2.6	.2	6.47	23.4	0.10	1.9
				Sewell Subdivision #4				
HV-1W	652	<1	15.1	1.4	0.01	4.1	0.04	NA
HV-2P	685	<1	5.1	.2	0.03	34	0.04	2.2
HV-3W	715	<1	8.4	1.2	0.86	24	0.05	NA
HV-4P	NS							
HV-5W	736	<1	7.1	1.0	0.01	8.5	0.41	NA
HV-6P	NS							
HV-7W	699	<1	7.8	1.4	3.21	4.5	0.08	NA
HV-8P	913	<1	5.9	.2	0.24	0	0.04	1.4

APPENDIX

SUBDIVISION

GROUNDWATER PROJECT RESULTS

SUBDIVISION: Five Acre Tracts

DATE: ____________

SITE	pH	SPECIFIC CONDUCTANCE (micromhos per cm.)	F.C. (per 100 mls.)	CL^- mg/l	NO_3 mg/l	NH_3 mg/l	COD mg/l	PO_4-P mg/l	GROUND-WATER DEPTH (ft.)
DATE: 9/26/78									
LV-1 W	7.5	NS	< 1	6.20	0.80	ND	16	ND	-
LV-2 W	7.6	NS	< 1	6.13	3.40	1.50	11	0.07	-
LV-3 W	7.3	NS	< 1	6.10	1.20	ND	14	0.04	-
LV-4a W	7.6	NS	< 1	6.19	0.50	ND	20	ND	-
LV-4b W	7.6	NS	< 1	6.69	0.50	ND	10	0.04	-
LV-5 W	7.6	NS	< 1	5.88	0.80	ND	8	0.02	-
LV-6 P	NS	NS	NS	NS	NS	NS	NS	NS	> 6.0'
LV-7 P	7.5	NS	< 1	7.26	2.20	ND	18	1.1	@3.0'
LV-8 W	7.5	NS	< 1	5.94	NS	ND	NS	NS	-
LV-9 P	NS	-	-	-	-	-	-	-	> 11.0'
DATE: 10/26/78									
LV-1 W	7.0	502	<1	6.82	0.60	ND	2	0.11	-
LV-2 W	NS	-	-	-	-	-	-	-	-
LV-3 W	6.7	520	< 1	6.69	0.90	ND	ND	0.11	-
LV-4a W	6.7	353	< 1	7.39	0.30	ND	ND	0.11	-
LV-4b W	6.7	400	< 1	6.73	0.40	ND	ND	0.12	-
LV-5 W	7.0	400	< 1	6.51	0.50	ND	6.0	0.19	-
LV-6 P	NS	-	-	-	-	-	-	-	>6.0'
LV-7 P	NS	-	-	-	-	-	-	-	-
LV-8 W	7.2	357	< 1	7.22	0.50	ND	7.0	0.12	-
LV-9 P	NS	-	-	-	-	-	-	-	> 11.0'
DATE: 2/9/78									
LV-1 W	6.9	450	<1	7.38	0.32	ND	NS	0.05	-
LV-2 W	NS	-	-	-	-	-	-	-	-
LV-3 W	7.1	430	<1	7.27	0.50	ND	NS	0.03	-
LV-4b W	7.1	370	<1	7.96	0.47	ND	NS	0.05	-
LV-5 W	7.2	390	<1	6.11	0.28	ND	NS	0.04	-
LV-6 P	NS	-	-	-	-	-	-	-	>6.0'
LV-7 P	NS	-	-	-	-	-	-	-	>4.2'
LV-8 W	NS	-	-	-	-	-	-	-	-
LV-9 P	NS	-	-	-	-	-	-	-	>11.0'

NS: NO SAMPLE AVAILABLE
ND: NOT DETECTABLE (< 0.01)
NC: ANALYSIS NOT CONDUCTED

*: NEW SITE SUBSTITUTE

SUBDIVISION

GROUNDWATER PROJECT RESULTS

SUBDIVISION: FIVE ACRE TRACTS

DATE:

SITE	pH	SPECIFIC CONDUCTANCE (micromhos per cm.)	F.C. (per 100 mls)	Cl^- mg/l	NO_3 mg/l	NH_3 mg/l	COD mg/l	PO_4-P mg/l	GROUND-WATER DEPTH (ft)
DATE: 3/23/79									
LV-1 W	7.35	457	NS	6.46	0.23	ND	17.5	0.05	-
LV-2 W	NS	-	-	-	-	-	-	-	-
LV-3 W	7.33	502	NS	12.95	0.38	ND	ND	0.04	-
LV-4b W	7.38	450	NS	15.45	0.40	ND	ND	0.05	-
LV-5 W	7.45	448	NS	6.58	0.23	ND	ND	0.07	-
LV-6 P	NS	-	-	-	-	-	-	-	6.0'
LV-7 P	NS	-	-	-	-	-	-	-	4.2'
LV-8 P	7.36	379	NS	9.65	0.74	ND	ND	0.23	-
LV-9 P	NS	-	-	-	-	-	-	-	11.0'
DATE: 4/21/79									
LV-1 W	NS	-	-	-	-	-	-	-	-
LV-2 W	-	-	-	-	-	-	-	-	-
LV-3 W	7.90	551	<1	19.70	0.62	ND	24.0	0.10	-
LV-4b W	7.60	442	<1	7.56	0.60	0.03	23.6	0.05	-
LV-5 W	7.40	478	<1	5.58	0.48	ND	23.6	0.07	-
LV-6 P	NS	-	-	-	-	-	-	-	6.0'
LV-7 P	NS	-	-	-	-	-	-	-	4.2'
LV-9 P	NS	-	-	-	-	-	-	-	11.0'
*LV-10 W	7.15	368		6.73	0.56	ND	24.0	0.05	-
DATE: 6/14/79									
LV-1 P	7.2	564	<1	7.67	1.50	ND	8.8	0.03	-
*LV-2 W	7.2	242	<1	5.10	0.32	0.04	2.2	0.03	-
LV-3 W	7.3	530	<1	20.90	0.70	0.01	NS	0.03	-
LV-4a W	7.4	429	<1	7.38	0.59	0.02	NS	0.03	-
LV-4b W	7.4	391	<1	8.06	0.50	0.02	ND	0.04	-
LV-5 W	7.7	450	<1	11.14	0.58	0.02	34.7	0.03	-
LV-6 P	NS	-	-	-	-	-	-	-	6.0'
LV-7 P	NS	-	-	-	-	-	-	-	4.2'
LV-9 P	NS	-	-	-	-	-	-	-	11.
*LV-10 W	7.3	380	<1	8.90	0.76	ND	57.1	0.03	-

NS: NO SAMPLE AVAILABLE

ND: NOT DETECTABLE (< 0.01)

NC: ANALYSIS NOT CONDUCTED

*: NEW SITE SUBSTITUTE

SUBDIVISION

GROUNDWATER PROJECT RESULTS

SUBDIVISION: FIVE ACRE TRACTS

DATE:

SITE	pH	SPECIFIC CONDUCTANCE (micromhos per cm.)	F.C. (per 100 mls)	Cl^- mg/l	NO_3 mg/l	NH_3 mg/l	COD mg/l	PO_4-P mg/l	GROUND-WATER DEPTH (ft)
DATE: 6/29/79									
LV-1 W	7.4	632	< 1	0.3	1.4	.03	23.2	0.34	N/A
*LV-2 W	7.3	351	< 1	1.2	1.7	ND	21.1	0.10	N/A
LV-3 W	7.2	614	< 1	0.3	1.0	0.03	23.3	0.10	N/A
LV-4b W	7.4	416	< 1	0.5	0.80	0.40	22.3	0.30	N/A
LV-5 W	NS	NS	-	-	-	-	-	-	-
LV-6 P	NS	NS	-	-	-	-	-	-	-
LV-7 P	7.5	197	1	0.9	0.5	0.10	22.4	ND	2.6
LV-9 P	NS	NS	-	-	-	-	-	-	-
*LV-10 W	7.3	460	< 1	0.2	1.1	ND	23.2	0.10	N/A
DATE: 7/14/79									
LV-1 W	7.0	607	<.1	1.09	0.54	ND	32.64	0.04	N/A
*LV-2 W	7.2	429	<.1	1.30	0.10	ND	20.40	0.06	N/A
LV-3 W	7.1	607	<1	1.30	0.44	ND	16.32	ND	N/A
LV-4 W	7.2	429	<1	1.22	0.66	ND	6.12	ND	N/A
LV-5 W	NS	-	-	-	-	-	-	-	-
LV-6 P	NS	DRY	-	-	-	-	-	-	-
LV-7 P	7.1	209	< 1	4.47	ND	ND	24.48	0.02	-
LV-8 W	NS	-	-	-	-	-	-	-	-
LV-9 P	NS	DRY	-	-	-	-	-	-	-
*LV-10 W	7.2	419	<1	1.95	0.31	ND	ND	0.06	-
DATE: 7/27/79									
LV-1 W	NS	NS	< 1	1.34	1.4	ND	20.6	ND	N/A
*LV-2 W	NS	NS	< 1	2.02	1.0	ND	22.4	0.06	N/A
LV-3 W	NS	NS	< 1	1.34	0.9	ND	21.4	ND	N/A
LV-4 W	NS	NS	< 1	1.34	0.7	ND	23.4	ND	N/A
LV-5 W	NS	-	-	-	-	-	-	-	-
LV-6 P	NS	NS	< 1	1.12	0.67	ND	22.8	0.04	5.5
LV-7 P	NS	NS	< 1	4.26	0.67	ND	23.2	ND	3.9
LV-8 W	NS	-	-	-	-	-	-	-	-
LV-9 P	NS	DRY	-	-	-	-	-	-	-
*LV-10 W	NS	NS	<1	2.02	0.97	ND	23.3	ND	N/A

NS: NO SAMPLE AVAILABLE

ND: NOT DETECTABLE (< 0.01)

NC: ANALYSIS NOT CONDUCTED

*: NEW SITE SUBSTITUTE

SUBDIVISION

GROUNDWATER PROJECT RESULTS

SUBDIVISION: FIVE ACRE TRACTS

DATE: ____________

SITE	pH	SPECIFIC CONDUCTANCE (micromhos per cm.)	F.C. (per 100 mls)	Cl^- mg/l	NO_3 mg/l	NH_3 mg/l	COD mg/l	PO_4-P mg/l	GROUND-WATER DEPTH (ft)
DATE: 8/17/79									
LV-1 P	7.4	681	<1	0.84	0.90	ND	95	0.04	N/A
*LV-2 W	NS	-	-	-	-	-	-	-	-
LV-3W	7.5	597	<1	1.70	0.51	ND	21	ND	N/A
LV-4 W	7.8	429	<1	0.84	0.28	ND	62	0.1	N/A
LV-5 W	7.0	785	2	0.31	NS	69.3	NS	ND	N/A
LV-6 P	7.8	419	3	0.97	0.22	ND	4	0.08	5.3
LV-7 P	7.0	269	<1	0.78	ND	ND	ND	0.12	1.7
LV-8 W	NS	-	-	-	-	-	-	-	-
LV-9 P	DRY	-	-	-	-	-	-	-	-
*LV-10 W	7.7	440	<1	0.84	0.38	ND	ND	0.06	N/A
DATE:									
DATE:									

NS: NO SAMPLE AVAILABLE
ND: NOT DETECTABLE (< 0.01)
NC: ANALYSIS NOT CONDUCTED

*: NEW SITE SUBSTITUTE

SUBDIVISION

GROUNDWATER PROJECT RESULTS

SUBDIVISION: Lambkin Addition

DATE:

SITE	pH	SPECIFIC CONDUCTANCE (micromhos per cm.)	F.C. (per 100 mls)	Cl$^-$ mg/l	NO$_3$ mg/l	NH$_3$ mg/l	COD mg/l	PO$_4$-P mg/l	GROUND-WATER DEPTH (ft)
DATE: 10/27/78									
LN-1 P	NS	-	-	-	-	-	-	-	-
LN-2 P	NS	-	-	-	-	-	-	-	@ 3.6'
LN-3 W	NC	342	<1	1.54	0.10	ND	ND	0.10	-
LN-4 W	NC	351	<1	1.50	0.30	ND	ND	0.10	-
LN-5 P	NS	-	-	-	-	-	-	-	
LN-6 W	NC	323	<1	1.67	0.20	ND	ND	0.10	-
LN-7 P	NS	-	-	-	-	-	-	-	
LN-8 W	NC	330	<1	1.58	0.20	ND	25	0.09	-
LN-9 P	NS	-	-	-	-	-	-	-	-
DATE: 1/13/79									
LN-1 P	NS	-	-	-	-	-	-	-	@ 11.6
LN-2 P	7.4	315	<1	1.69	0.22	ND	3	0.23	@ 5.0'
LN-3 W	7.6	325	<1	2.15	0.22	ND	ND	0.08	-
LN-4 W	7.3	331	<1	1.98	0.30	ND	ND	0.11	-
LN-5 P	NS	-	-	-	-	-	-	-	@ 7.9'
LN-6 W	7.6	330	<1	1.98	0.31	ND	ND	0.08	-
LN-7 P	7.4	344	<1	1.94	0.31	ND	18	0.34	@ 4.2'
LN-8 W	7.4	330	<1	1.98	0.31	ND	5	0.08	-
LN-9 P	NS	-	-	-	-	-	-	-	@ 2.1'
DATE: 4/21/79									
LN-1 P	7.4	333	3	3.78	0.19	ND	16.7	ND	@ 6.3'
LN-2 P	7.2	348	<1	2.88	5.40	ND	ND	ND	@ 5.1'
LN-3 W	7.6	308	<1	2.36	ND	ND	ND	ND	-
LN-4 W	7.4	320	<1	2.10	0.19	ND	ND	ND	-
LN-5 P	7.2	331	<1	2.93	0.11	ND	ND	ND	@ 4.2'
LN-6 W	7.4	312	<1	2.06	0.16	ND	ND	ND	-
LN-7 P	7.3	371	<1	4.25	0.57	ND	10.5	ND	@ 4.2'
LN-8 P	7.4	329	<1	2.19	0.25	ND	ND	ND	-
LN-9 W	7.9	391	<1	5.44	0.26	ND	87.9	ND	@ 3.2'

NS: NO SAMPLE AVAILABLE
ND: NOT DETECTABLE (< 0.01)
NC: ANALYSIS NOT CONDUCTED

*: NEW SITE SUBSTITUTE

SUBDIVISION

GROUNDWATER PROJECT RESULTS

SUBDIVISION: Lambkin Addition

DATE:

SITE	pH	SPECIFIC CONDUCTANCE (micromhos per cm.)	F.C. (per 100 mls)	Cl$^-$ mg/l	NO$_3$ mg/l	NH$_3$ mg/l	COD mg/l	PO$_4$-P mg/l	GROUND-WATER DEPTH (ft)
DATE: 6/14/79									
LN-1 P	7.6	312	<1	2.50	0.30	0.03	6.35	0.06	NA
LN-2 P	7.2	340	<1	2.50	0.10	0.01	ND	0.04	@ 2.6'
LN-3 W	7.4	318	<1	3.03	0.40	0.04	ND	0.03	NA
LN-4 W	7.3	325	<1	3.03	0.34	0.04	ND	0.03	NA
LN-5 P	7.4	330	<1	2.95	0.30	0.05	ND	0.05	@ 5.0'
LN-6 W	7.4	315	<1	2.42	0.30	ND	ND	0.03	NA
LN-7 P	7.4	330	<1	3.19	0.27	0.02	ND	0.03	@ 2.6'
LN-8 W	7.4	320	<1	2.75	0.41	ND	ND	0.03	NA
LN-9 P	8.1	330	<1	3.51	0.38	0.2	ND	0.40	@ 1.6'
DATE: 7/13/79									
LN-1 P	NS	DRY	-	-	-	-	-	-	- -
LN-2 P	7.3	366	<1	2.82	ND	ND	22.4	0.02	
LN-3 W	7.4	419	<1	2.60	ND	ND	22.3	0.02	NA
LN-4 W	NS	-	-	-	-	-	-	-	-
LN-5 P	7.4	377	<1	2.82	ND	ND	23.4	ND	
LN-6 W	7.2	393	<1	2.08	ND	ND	23.4	ND	NA
LN-7 P	7.2	377	<1	2.82	ND	0.02	23.4	0.02	
LN-8 W	7.7	440	<1	3.04	ND	ND	23.3	ND	NA
LN-9 P	7.7	419	<1	2.08	ND	0.03	22.7	ND	
DATE: 7/28/79									
LN-1 P	7.7	387	<1	2.41	0.56	ND	19	ND	NA
LN-2 P	7.6	408	<1	1.79	0.54	ND	ND	ND	3.4
LN-3 W	7.8	387	<1	1.34	0.60	ND	36	ND	NA
LN-4 W	NS	-	-	-	-	-	-	-	-
LN-5 P	7.6	419	<1	1.34	0.58	ND	15	ND	3.5
LN-6 W	7.7	387	<1	2.24	0.60	ND	6	ND	NA
LN-7 P	7.8	429	<1	1.12	0.70	ND	ND	ND	3.3
LN-8 W	7.9	408	<1	0.90	0.70	ND	6	ND	NA
LN-9 P	8.5	461	<1	2.02	ND	ND	2	ND	0.6

NS: NO SAMPLE AVAILABLE

ND: NOT DETECTABLE (< 0.01)

NC: ANALYSIS NOT CONDUCTED

*: NEW SITE SUBSTITUTE

SUBDIVISION

GROUNDWATER PROJECT RESULTS

SUBDIVISION: Lambkin Addition

DATE:

SITE	pH	SPECIFIC CONDUCTANCE (micromhos per cm.)	F.C. (per 100 mls)	Cl^- mg/l	NO_3 mg/l	NH_3 mg/l	COD mg/l	PO_4-P mg/l	GROUND-WATER DEPTH (ft)
DATE: 8/19/79									
LN-1 P	7.8	366	<1	1.31	ND	ND	29.54	0.04	NA
LN-2 P	7.5	419	<1	0.26	ND	ND	59.0	0.04	1.2
LN-3 W	7.7	398	<1	0.16	ND	ND	ND	0.35	NA
LN-4 W	UNABLE TO GAIN ACCESS		-	-	-	-	-	-	-
LN-5 P	DRY	-	-	-	-	-	-	-	-
LN-6 W	7.7	461	<1	1.05	0.1	ND	12.66	ND	NA
LN-7 P	7.6	492	<1	0.34	0.11	ND	59.0	0.1	1.2
LN-8 W	7.7	387	<1	0.73	0.11	ND	4.20	ND	NA
LN-9 P	7.9	503	<1	0.13	ND	32.1	4.20	0.08	1.8
DATE:									
DATE:									

NS: NO SAMPLE AVAILABLE

ND: NOT DETECTABLE (< 0.01)

NC: ANALYSIS NOT CONDUCTED

*: NEW SITE SUBSTITUTE

SUBDIVISION

GROUNDWATER PROJECT RESULTS

SUBDIVISION: Sewell Subdivision #4

DATE:

SITE	pH	SPECIFIC CONDUCTANCE (micromhos per cm.)	F.C. (per 100 mls)	Cl^- mg/l	NO_3 mg/l	NH_3 mg/l	COD mg/l	PO_4-P mg/l	GROUND-WATER DEPTH (ft)
DATE: 10/27/78									
HV-1 W	NC	651	<1	13.3	1.60	ND	ND	NS	-
HV-2 P	NS	-	-	-	-	-	-	-	>8.1'
HV-3 W	NC	632	<1	13.2	1.70	ND	23	0.10	-
HV-4 P	NS	-	-	-	-	-	-	-	>8.5'
HV-5 W	NC	672	<1	12.2	1.60	ND	12	0.11	-
HV-6 P	NS	-	-	-	-	-	-	-	>8.2'
HV-7 W	NC	603	<1	12.3	2.20	ND	ND	0.11	-
HV-8 P	NS	-	-	-	-	-	-	-	>7.5'
DATE: 1/13/79									
HV-1 W	7.2	658	<1	15.8	1.25	ND	ND	0.11	-
HV-2 P	NS	-	-	-	-	-	-	-	>8.1'
HV-3 W	7.2	641	<1	13.5	0.98	ND	ND	0.11	-
HV-4 P	NS	-	-	-	-	-	-	-	>8.5'
HV-5 W	NS	-	-	-	-	-	-	-	-
HV-6 P	NS	-	-	-	-	-	-	-	>8.2'
HV-7 W	7.1	615	<1	12.9	1.60	ND	ND	0.11	-
HV-8 P	NS	-	-	-	-	-	-	-	>7.5'
DATE: 4/21/79									
HV-1 W	7.5	650	<1	17.8	1.06	ND	ND	ND	
HV-2 P	NS	-	-	-	-	-	-	-	@6.9'
HV-3 W	7.4	652	<1	14.2	1.00	ND	ND	0.12	-
HV-4 P	NS	-	-	-	-	-	-	-	@6.0'
HV-5 W	7.2	680	<1	14.0	0.74	ND	ND	0.03	-
HV-6 P	NS	-	-	-	-	-	-	-	@6.9'
HV-7 W	7.2	620	<1	13.2	0.95	0.02	ND	0.20	-
HV-8 P	NS	-	-	-	-	-	-	-	@4.8'

NS: NO SAMPLE AVAILABLE
ND: NOT DETECTABLE (< 0.01)
NC: ANALYSIS NOT CONDUCTED

*: NEW SITE SUBSTITUTE

SUBDIVISION

GROUNDWATER PROJECT RESULTS

SUBDIVISION: Sewell Subdivision #4

DATE:

SITE	pH	SPECIFIC CONDUCTANCE (micromhos per cm.)	F.C. (per 100 mls)	Cl^- mg/l	NO_3 mg/l	NH_3 mg/l	COD mg/l	PO_4-P mg/l	GROUND-WATER DEPTH (ft)
DATE: 6/14/79									
HV-1 W	7.3	650	<1	13.41	1.50	0.04	16.2	0.03	-
HV-2 P	7.6	382	<1	12.28	0.16	0.03	48.7	0.06	@2.6'
HV-3 W	7.3	640	<1	12.60	1.00	0.04	80.6	0.03	-
HV-4 P	NS	-	-	-	-	-	-	-	@6.1'
HV-5 W	7.3	665	<1	12.12	1.00	0.01	18.6	0.03	-
HV-6 P	NS	-	-	-	-	-	-	-	@4.8'
HV-7 W	7.2	615	<1	12.16	1.70	0.03	NC	0.03	-
HV-8 P	9.1	675	<1	10.42	0.35	0.42	ND	0.06	@1.4'
DATE: 7/13/79									
HV-1 W	PUMP ON WELL BROKEN		-	-	-	-	-	-	-
HV-2 P	7.9	722	<1	3.04	0.44	0.09	19.2	ND	
HV-3 W	7.6	733	<1	1.74	1.4	6.0	18.0	ND	NA
HV-4 P	DRY	-	-	-	-	-	-	-	-
HV-5 W	7.3	827	<1	2.39	0.46	0.04	20.5	0.04	NA
HV-6 P	DRY	-	-	-	-	-	-	-	-
HV-7 W	7.2	733	<1	1.95	1.4	0.05	23.4	ND	NA
HV-8 P	9.1	1152	<1	1.30	<0.1	0.06	ND.	0.02	
DATE: 7/28/79									
HV-1 W	PUMP ON WELL BROKEN		-	-	-	-	-	-	-
HV-2 P	NS	NS	<1	4.92	0.14	ND	ND	0.06	NA
HV-3 W	NS	NS	<1	2.24	1.4	ND	11	ND	NA
HV-4 P	DRY	-	-	-	-	-	-	-	-
HV-5 W	NS	NS	<1	1.57	1.6	ND	ND	0.02	NA
HV-6 P	DRY	-	-	-	-	-	-	-	-
HV-7 W	NS	NS	<1	1.90	1.8	22.4	ND	0.02	NA
HV-8 P	DRY	-	-	-	-	-	-	-	-

NS: NO SAMPLE AVAILABLE

ND: NOT DETECTABLE (< 0.01)

NC: ANALYSIS NOT CONDUCTED

*: NEW SITE SUBSTITUTE

SUBDIVISION

GROUNDWATER PROJECT RESULTS

SUBDIVISION: Sewell Subdivision #4

DATE:

SITE	pH	SPECIFIC CONDUCTANCE (micromhos per cm.)	F.C. (per 100 mls)	Cl^- mg/l	NO_3 mg/l	NH_3 mg/l	COD mg/l	PO_4-P mg/l	GROUND-WATER DEPTH (ft)
DATE: 8/19/79									
HV-1 W	PUMP BROKEN		-	-	-	-	-	-	-
HV-2 P	7.3	953	<1	0.26	0.21	ND	ND	0.02	1.9'
HV-3 W	7.5	995	<1	1.49	0.80	ND	ND	ND	NA
HV-4 P	DRY	-	-	-	-	-	-	-	-
HV-5 W	7.7	838	<1	0.29	0.70	ND	ND	2.20	NA
HV-6 P	DRY	-	-	1.38	-	-	-	-	-
HV-7 W	7.5	1005	<1	0.26	<.1	ND	8	0.06	NA
HV-8 P	DRY	-	-	1.31	-	-	-	-	-
DATE:									
DATE:									

NS: NO SAMPLE AVAILABLE
ND: NOT DETECTABLE (< 0.01)
NC: ANALYSIS NOT CONDUCTED

*: NEW SITE SUBSTITUTE

SUBDIVISION

GROUNDWATER PROJECT RESULTS

SUBDIVISION: Entropy Acres

DATE:

SITE	pH	SPECIFIC CONDUCTANCE (micromhos per cm.)	F.C. (per 100 mls)	Cl^- mg/l	NO_3 mg/l	NH_3 mg/l	COD mg/l	PO_4-P mg/l	GROUND-WATER DEPTH (ft)
DATE: 3/21/79									
HD-1 P	NS	-	-	-	-	-	-	-	>9.2'
HD-2 W	7.2	319	<1	2.19	0.79	ND	ND	0.09	
HD-3 P	NS	-	-	-	-	-	-	-	>6.1'
HD-4 W	7.3	340	<1	1.45	0.33	ND	ND	0.04	
HD-5 P	NS	-	-	-	-	-	-	-	>2.9'
HD-6 W	7.2	353	<1	1.79	0.38	ND	ND	0.04	
HD-7 P	NS	-	-	-	-	-	-	-	>6.4'
HD-8 W	7.4	310	<1	1.31	0.28	ND	ND	0.02	
DATE: 6/21/79									
HD-1 P	7.0	333	<1	4.61	1.40	ND	22.0	0.10	@3.2'
HD-2 W	7.1	338	<1	4.24	1.10	ND	ND	0.06	
HD-3 P	7.1	238	<1	3.59	0.74	0.625	22.7	0.05	@7.0'
HD-4 W	7.0	371	<1	3.65	0.72	0.040	ND	0.02	
HD-5 P	7.3	184	<1	3.06	0.66	0.045	22.0	ND	@6.6'
HD-6 W	7.1	375	<1	3.98	0.84	ND	ND	0.06	
HD-7 P	7.1	241	<1	3.65	0.74	ND	22.0	0.02	@6.1'
HD-8 W	7.1	350	<1	3.73	0.70	0.060	ND	NS	
DATE: 6/30/79									
HD-1 P	6.9	565	<1	0.3	0.96	0.09	ND	0.18	3.2'
HD-2 W	7.2	524	<1	0.2	1.9	ND	ND	0.16	NA
HD-3 P	7.0	440	<1	0.4	1.1	0.03	ND	0.14	7.2'
HD-4 W	6.9	524	<1	0.3	1.2	ND	ND	ND	NA
HD-5 P	6.8	419	<1	0.7	0.9	ND	ND	0.10	6.4'
HD-6 W	6.9	524	<1	0.8	0.9	ND	ND	0.06	NA
HD-7 P	7.0	440	<1	0.6	0.96	0.03	ND	0.28	7.2'
HD-8 W	7.1	520	<1	0.3	0.84	0.03	ND	ND	NA

NS: NO SAMPLE AVAILABLE

ND: NOT DETECTABLE (< 0.01)

NC: ANALYSIS NOT CONDUCTED

*: NEW SITE SUBSTITUTE

SUBDIVISION

GROUNDWATER PROJECT RESULTS

SUBDIVISION: Entropy Acres

DATE:

SITE	pH	SPECIFIC CONDUCTANCE (micromhos per cm.)	F.C. (per 100 mls)	Cl^- mg/l	NO_3 mg/l	NH_3 mg/l	COD mg/l	PO_4-P mg/l	GROUND-WATER DEPTH (ft)
DATE: 7/13/79									
HD-1 P	7.3	419	<1	1.30	0.1	ND	ND	0.24	2.3
HD-2 W	7.1	377	<1	0.87	0.53	0.09	10.33	0.04	NA
HD-3 P	7.0	304	<1	0.99	0.1	0.12	6.19	0.06	6.7
HD-4 W	6.9	346	<1	3.47	0.33	0.05	ND	ND	NA
HD-5 P	6.9	293	<1	1.30	0.33	ND	8.26	ND	6.3
HD-6 W	7.4	335	<1	1.52	0.32	0.01	8.26	ND	NA
HD-7 P	7.3	319	<1	1.09	0.33	ND	8.26	0.08	7.5
HD-8 W	7.1	366	<1	0.99	0.36	ND	30.99	ND	NA
DATE: 7/28/79									
HD-1 P	7.1	376	<1	1.57	0.62	ND	ND	0.02	2.3
HD-2 W	7.2	366	<1	1.57	0.94	ND	ND	ND	NA
HD-3 P	7.1	251	<1	1.34	0.87	ND	ND	ND	6.4
HD-4 W	7.1	325	<1	1.34	0.97	ND	ND	ND	NA
HD-5 P	7.1	309	<1	1.57	0.95	ND	ND	0.04	6.1
HD-6 W	7.3	304	<1	1.34	0.96	ND	ND	ND	NA
HD-7 P	7.0	293	<1	1.34	0.92	ND	ND	ND	7.3
HD-8 W	7.4	325	<1	0.90	0.96	ND	ND	ND	NA
DATE: 8/18/79									
HD-1 P	7.2	387	<1	0.89	0.38	10.0	13.0	0.28	2.7
HD-2 W	7.4	408	<1	0.26	0.40	ND	ND	0.10	NA
HD-3 P	7.4	262	<1	0.76	<0.1	ND	17	0.22	6.3
HD-4 W	7.5	377	<1	0.50	0.36	0.01	ND	0.02	NA
HD-5 P	7.2	272	<1	0.29	0.17	ND	13	0.14	5.8
HD-6 W	7.4	346	<1	0.26	0.38	10.0	ND	0.10	NA
HD-7 P	7.2	272	<1	0.24	0.17	ND	ND	0.08	7.0
HD-8 W	7.4	429	<1	0.63	0.29	ND	ND	0.08	NA

NS: NO SAMPLE AVAILABLE
ND: NOT DETECTABLE (< 0.01)
NC: ANALYSIS NOT CONDUCTED

*: NEW SITE SUBSTITUTE

SUBDIVISION

GROUNDWATER PROJECT RESULTS

SUBDIVISION: Hawker Lane East

DATE:

SITE	pH	SPECIFIC CONDUCTANCE (micromhos per cm.)	F.C. (per 100 mls)	Cl^- mg/l	NO_3 mg/l	NH_3 mg/l	COD mg/l	PO_4-P mg/l	GROUND-WATER DEPTH (ft)
DATE: 11/18/78									
HM-1 P	7.0	154	<1	3.24	ND	ND	ND	0.05	@5.4
HM-2 W	7.3	210	<1	3.29	ND	ND	4.0	ND	-
HM-3 P	7.2	151	<1	3.32	ND	ND	8.0	0.02	@6.7
HM-4 W	7.0	154	<1	3.40	ND	ND	12.0	0.01	-
HM-5 W	7.4	245	<1	3.19	ND	ND	2.0	ND	-
HM-6 W	7.1	153	<1	2.23	ND	ND	ND	ND	-
HM-7 P	7.1	205	<1	2.73	ND	ND	ND	0.03	@3.6
DATE: 3/21/79									
HM-1 P	6.7	187	<1	2.55	0.36	0.01	29.3	0.01	@6.6
HM-2 W	6.8	200	<1	1.93	0.36	ND	ND	0.03	-
HM-3 P	6.8	184	<1	4.98	0.20	0.02	35.5	0.12	@7.8
HM-4 W	6.9	168	<1	1.89	0.28	ND	ND	0.08	-
HM-5 W	7.1	180	<1	1.93	0.23	ND	ND	0.08	-
HM-6 W	7.1	722	<1	1.24	0.14	ND	ND	0.05	-
HM-7 P	NS	-	-	-	-	-	-	-	@4.0
DATE: 6/2/79									
HM-1 P	6.6	78	<1	2.31	0.30	ND	22.7	0.06	@3.2
HM-2 W	6.5	221	<1	8.09	0.44	0.02	ND	0.70	-
HM-3 P	6.6	224	<1	3.17	0.84	0.03	23.4	0.06	@2.6
HM-4 W	6.5	155	<1	3.66	0.58	0.04	ND	0.04	-
HM-5 W	6.4	182	<1	3.92	0.90	0.05	22.5	0.70	-
HM-6 W	6.6	238	<1	4.10	0.45	0.03	ND	ND	-
HM-7 P	7.0	275	<1	4.83	0.46	0.09	22.4	0.10	@2.3

NS: NO SAMPLE AVAILABLE
ND: NOT DETECTABLE (< 0.01)
NC: ANALYSIS NOT CONDUCTED

*: NEW SITE SUBSTITUTE

SUBDIVISION

GROUNDWATER PROJECT RESULTS

SUBDIVISION: Hawker Lane Estates

DATE:

SITE	pH	SPECIFIC CONDUCTANCE (micromhos per cm.)	F.C. (per 100 mls)	Cl^- mg/l	NO_3 mg/l	NH_3 mg/l	COD mg/l	PO_4-P mg/l	GROUND-WATER DEPTH (ft)
DATE: 6/30/79									
HM-1 P	6.3	261	<1	0.4	0.8	0.03	ND	0.08	4.3
HM-2 W	7.0	346	<1	0.3	0.46	0.01	ND	0.04	NA
HM-3 P	6.5	230	<1	0.6	0.88	0.14	ND	0.06	-
HM-4 W	6.9	240	<1	0.4	0.4	ND	ND	0.08	NA
HM-5 W	6.4	272	<1	0.7	0.84	ND	ND	0.14	NA
HM-6 W	6.7	366	<1	0.6	0.5	ND	ND	0.02	NA
HM-7 P	6.6	335	<1	0.4	0.4	0.02	ND	0.22	-
DATE: 7/13/79									
HM-1 P	6.9	188	<1	1.74	ND	ND	4.32	0.02	4.6
HM-2 W	6.6	272	<1	1.74	0.1	0.04	ND	ND	NA
HM-3 P	7.0	162	<1	2.39	ND	0.01	ND	0.04	5.1
HM-4 W	6.5	178	<1	1.87	ND	ND	ND	0.10	NA
HM-5 W	6.4	194	<1	2.08	0.1	ND	8.64	0.06	NA
HM-6 W	6.6	257	<1	2.30	ND	0.06	4.32	ND	NA
HM-7 P	7.1	230	<1	2.17	ND	0.04	2.16	0.12	2.5
DATE: 7/28/79									
HM-1 P	7.0	144	2	2.46	0.11	ND	ND	ND	3.7
HM-2 W	7.1	220	<1	2.24	0.90	ND	ND	ND	NA
HM-3 P	6.7	142	<1	1.79	0.40	ND	ND	ND	4.7
HM-4 W	6.8	173	<1	2.02	0.92	ND	ND	ND	NA
HM-5 W	6.4	168	<1	1.79	0.94	ND	ND	ND	NA
HM-6 W	6.7	236	<1	1.34	0.83	ND	ND	ND	NA
HM-7 P	7.0	183	<1	1.79	0.30	ND	ND	ND	2.2

NS: NO SAMPLE AVAILABLE

ND: NOT DETECTABLE (< 0.01)

NC: ANALYSIS NOT CONDUCTED

*: NEW SITE SUBSTITUTE

SUBDIVISION

GROUNDWATER PROJECT RESULTS

SUBDIVISION: Hawker Lane Estates

DATE:

SITE	pH	SPECIFIC CONDUCTANCE (micromhos per cm.)	F.C. (per 100 mls)	Cl⁻ mg/1	NO_3 mg/1	NH_3 mg/1	COD mg/1	PO_4-P mg/1	GROUND-WATER DEPTH (ft)
DATE: 7/13/79									
HM-1 P	6.9	168	1	1.74	ND	ND	4.32	0.02	4.6
HM-2 W	6.6	272	1	1.74	0.1	0.04	ND	ND	NA
HM-3 P	7.0	162	1	2.39	ND	0.01	ND	0.04	5.1
HM-4 W	6.5	178	1	1.87	ND	ND	ND	0.10	NA
HM-5 W	6.4	194	1	2.08	0.1	ND	8.64	0.06	NA
HM-6 W	6.6	257	1	2.30	ND	0.06	4.32	ND	NA
HM-7 P	7.1	230	1	2.17	ND	0.04	2.16	0.12	2.5
DATE: 7/28/79									
HM-1 P	7.0	144	2	2.46	0.11	ND	ND	ND	3.7
HM-2 W	7.1	220	1	2.24	0.90	ND	ND	ND	NA
HM-3 P	6.7	142	1	1.79	0.40	ND	ND	ND	4.7
HM-4 W	6.8	173	1	2.02	0.92	ND	ND	ND	NA
HM-5 W	6.4	168	1	1.79		ND	ND	ND	NA
HM-6 W	6.7	236	1	1.34	0.83	ND	ND	ND	NA
HM-7 P	7.0	183	1	1.79	0.30	ND	ND	ND	2.2
DATE: 8/18/79									
HM-1 P	7.0	183	1	0.21	ND	ND	ND	0.08	3.7
HM-2 W	7.1	241	1	0.65	0.19	0.01	ND	0.16	NA
HM-3 P	6.8	151	1	0.31	0.10	ND	13	0.08	5.3
HM-4 W	6.7	199	1	[illegible]	0.16	ND	17	0.16	NA
HM-5 W	6.9	178	1	0.29	0.15	9.4	ND	0.10	NA
HM-6 W	7.1	272	1	0.47	0.21	ND	ND	0.10	NA
HM-7 P	7.0	257	1	0.26	ND	ND	ND	0.10	[illegible]

NS: NO SAMPLE AVAILABLE
ND: NOT DETECTABLE (< 0.01)
NC: ANALYSIS NOT COMPLETED

*: NEW SITE SUBSTITUTE

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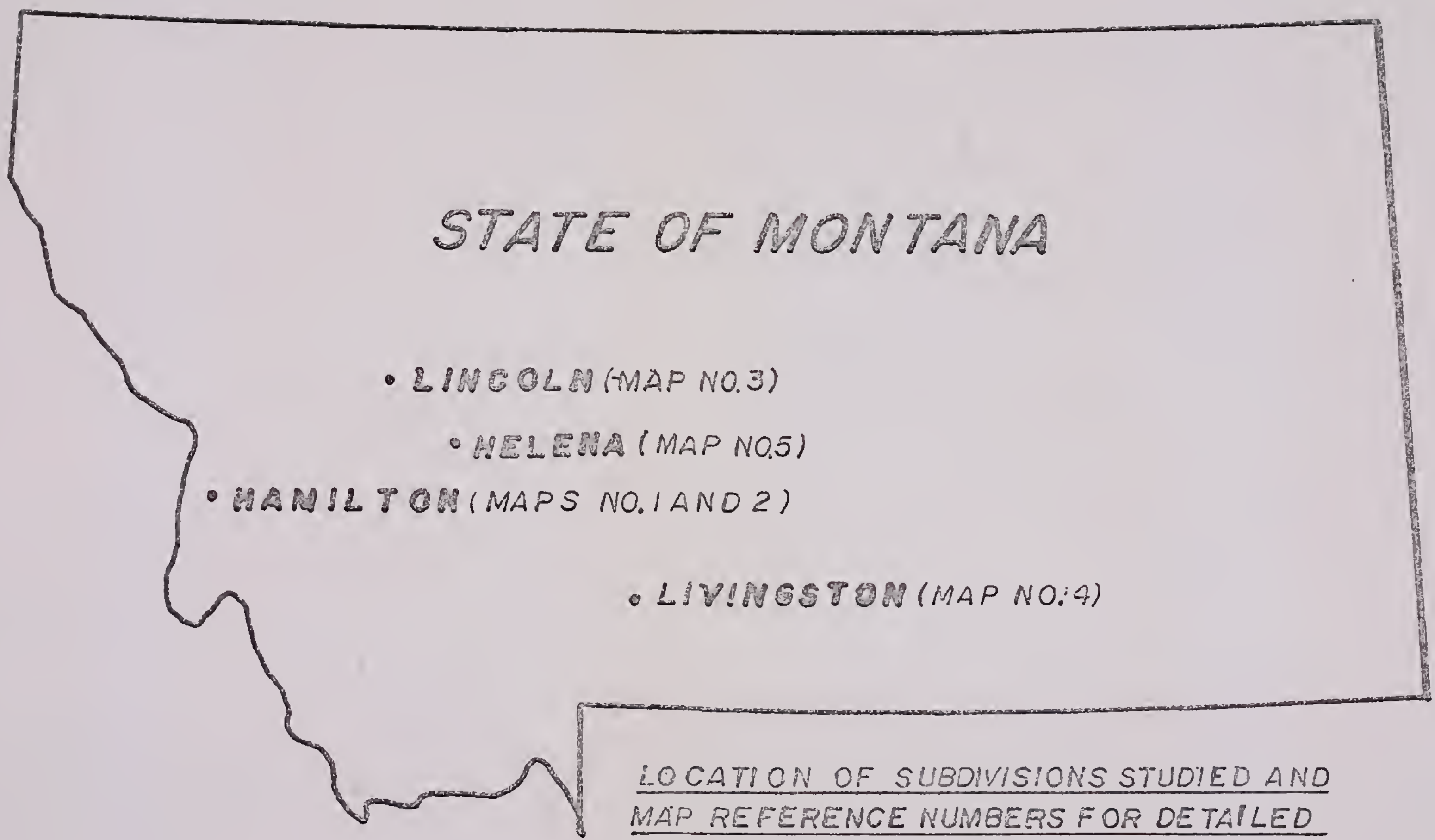
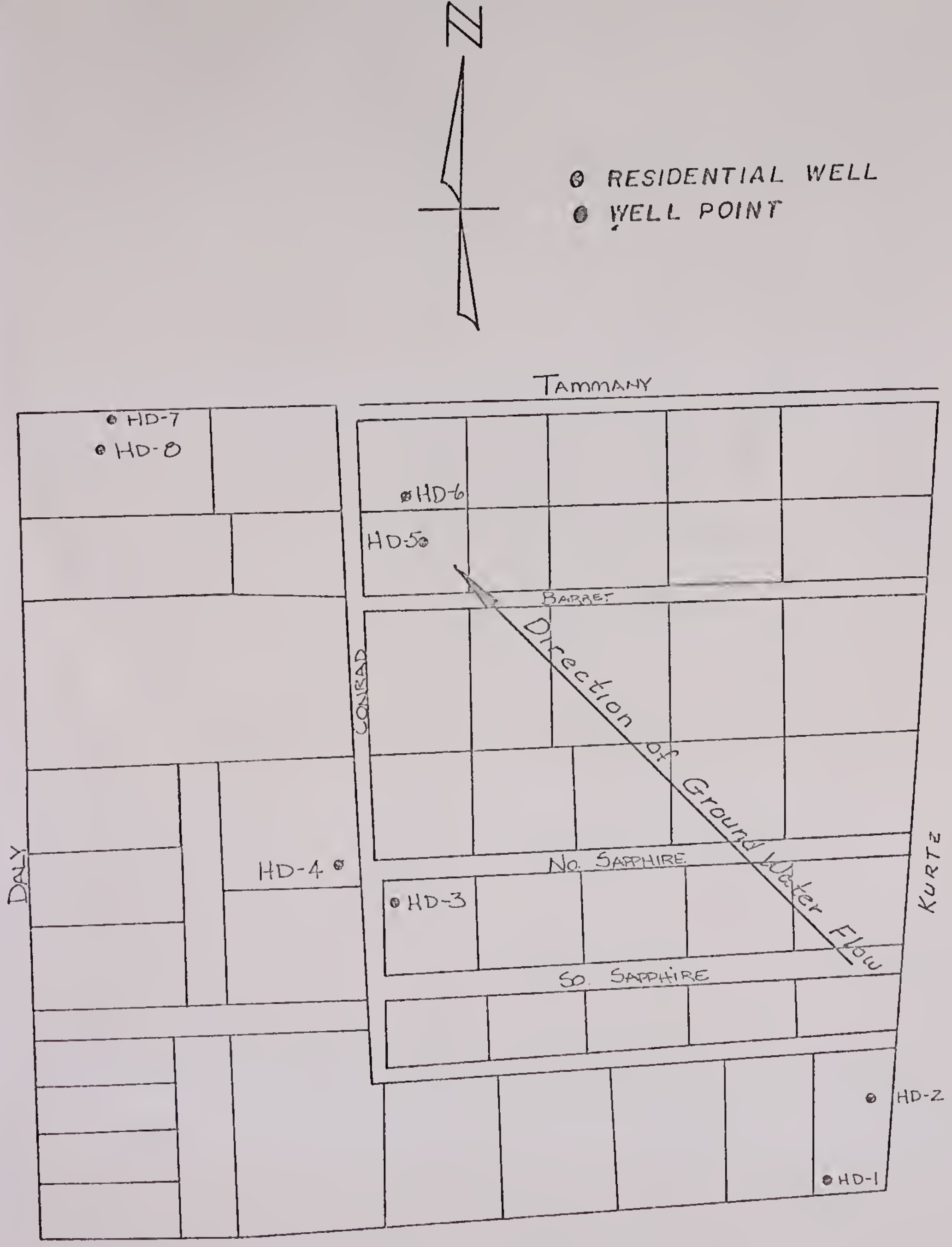
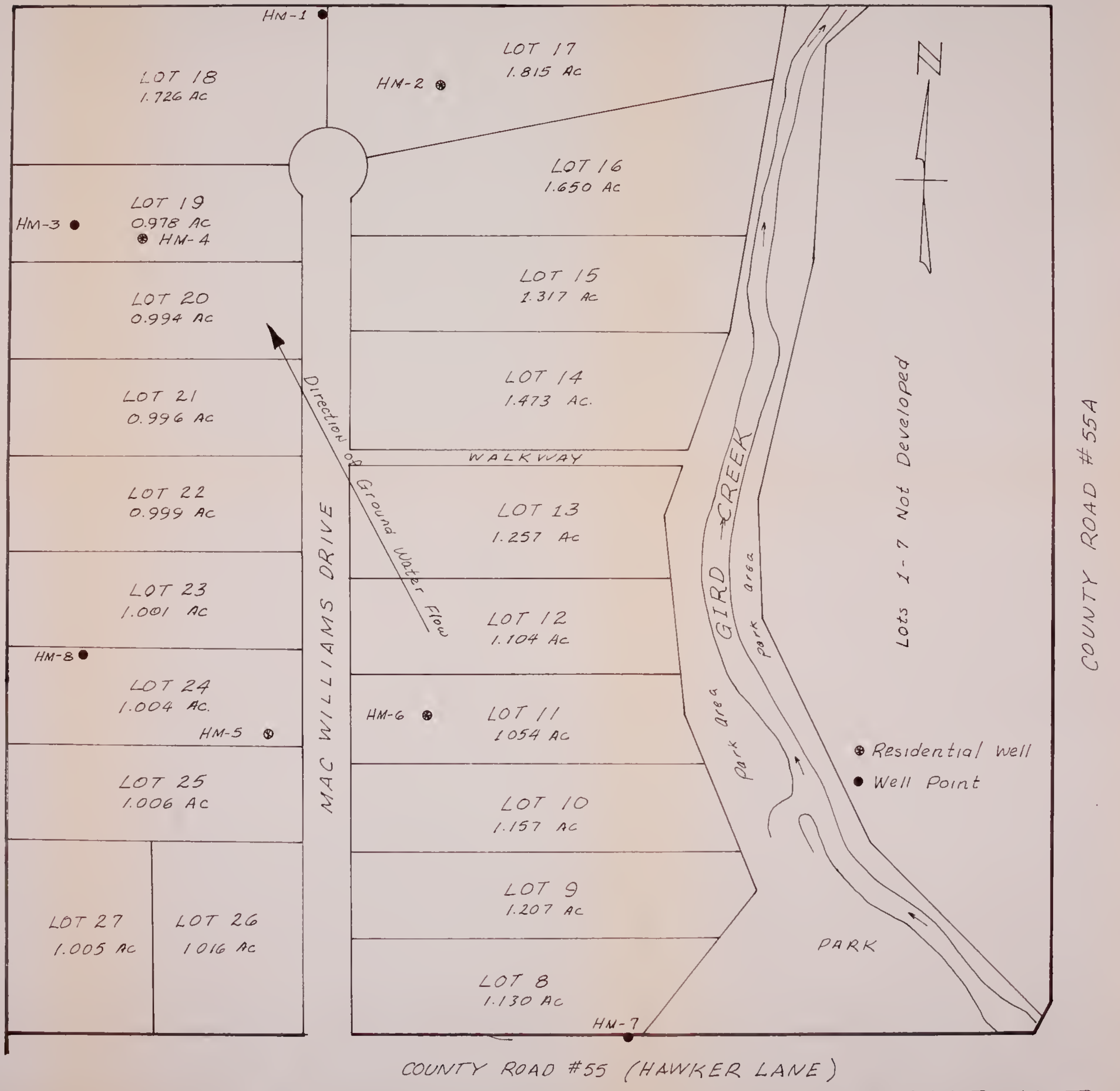


LOCATION OF SUBDIVISIONS STUDIED AND MAP REFERENCE NUMBERS FOR DETAILED SITE MAPS



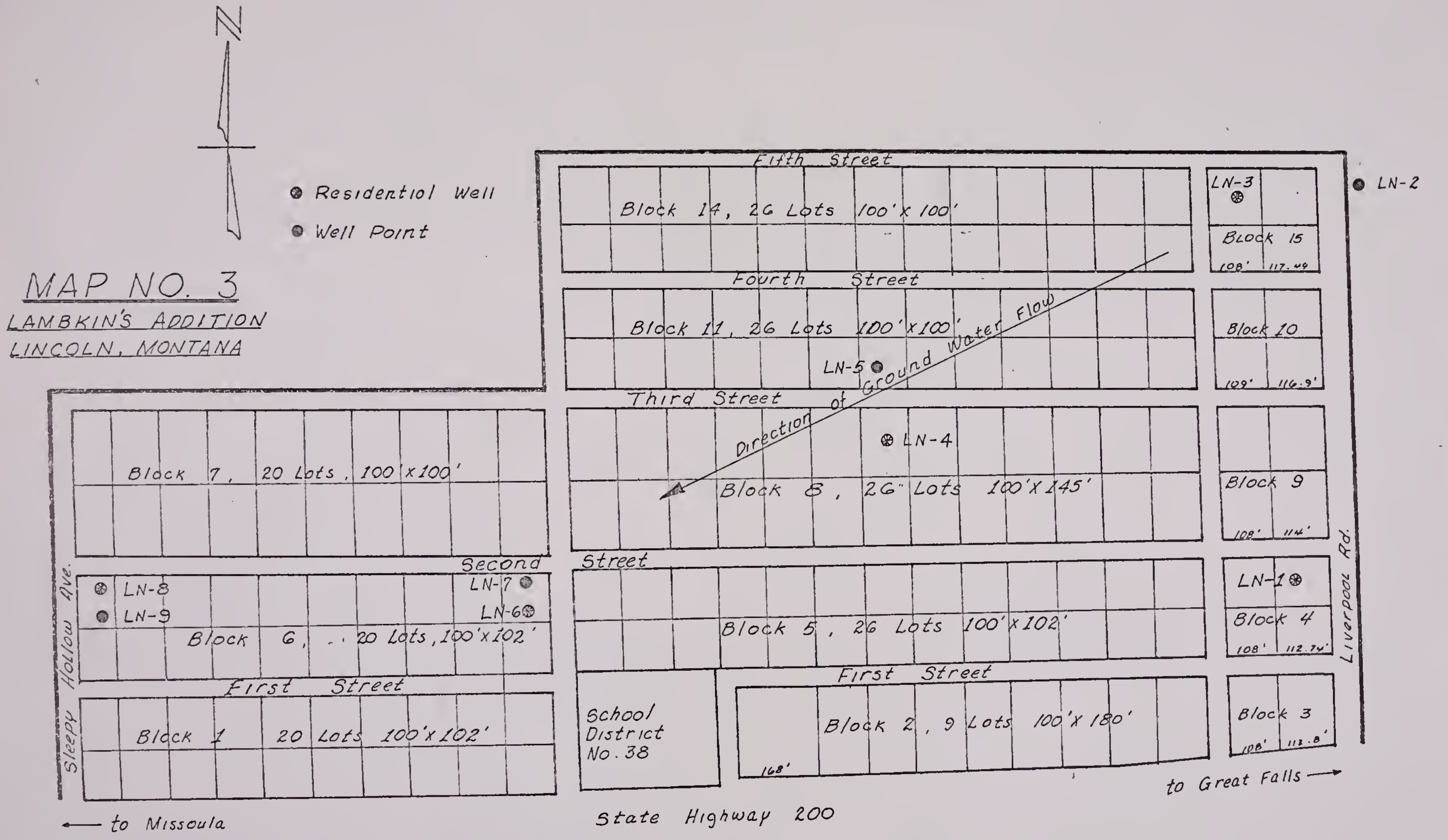
MAP NO. ONE

ENTROPY ACRES
HAMILTON, MONTANA



HAWKER LANE ESTATES
HAMILTON, MONTANA

MAP NO. TWO



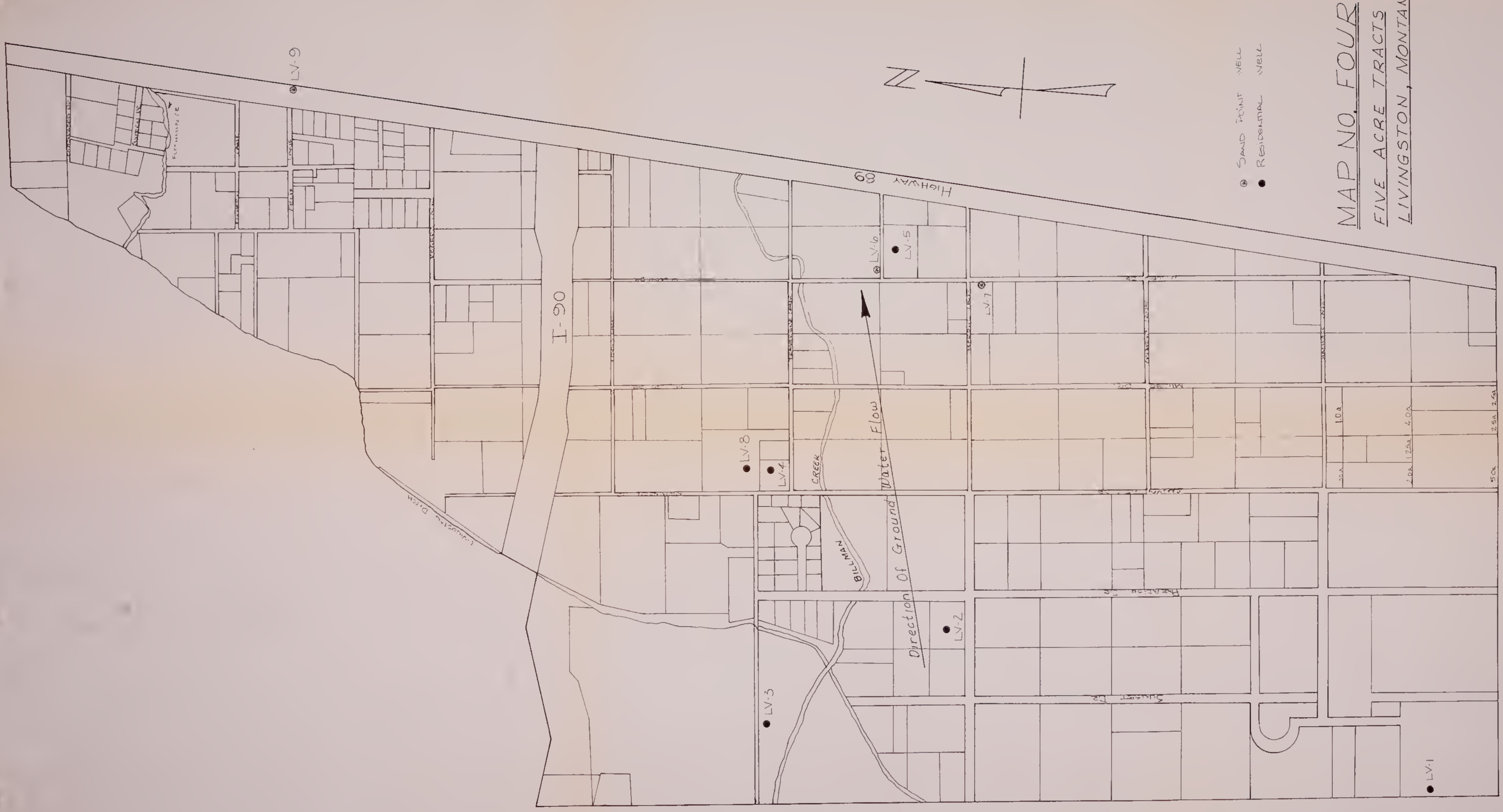
MAP NO. 3

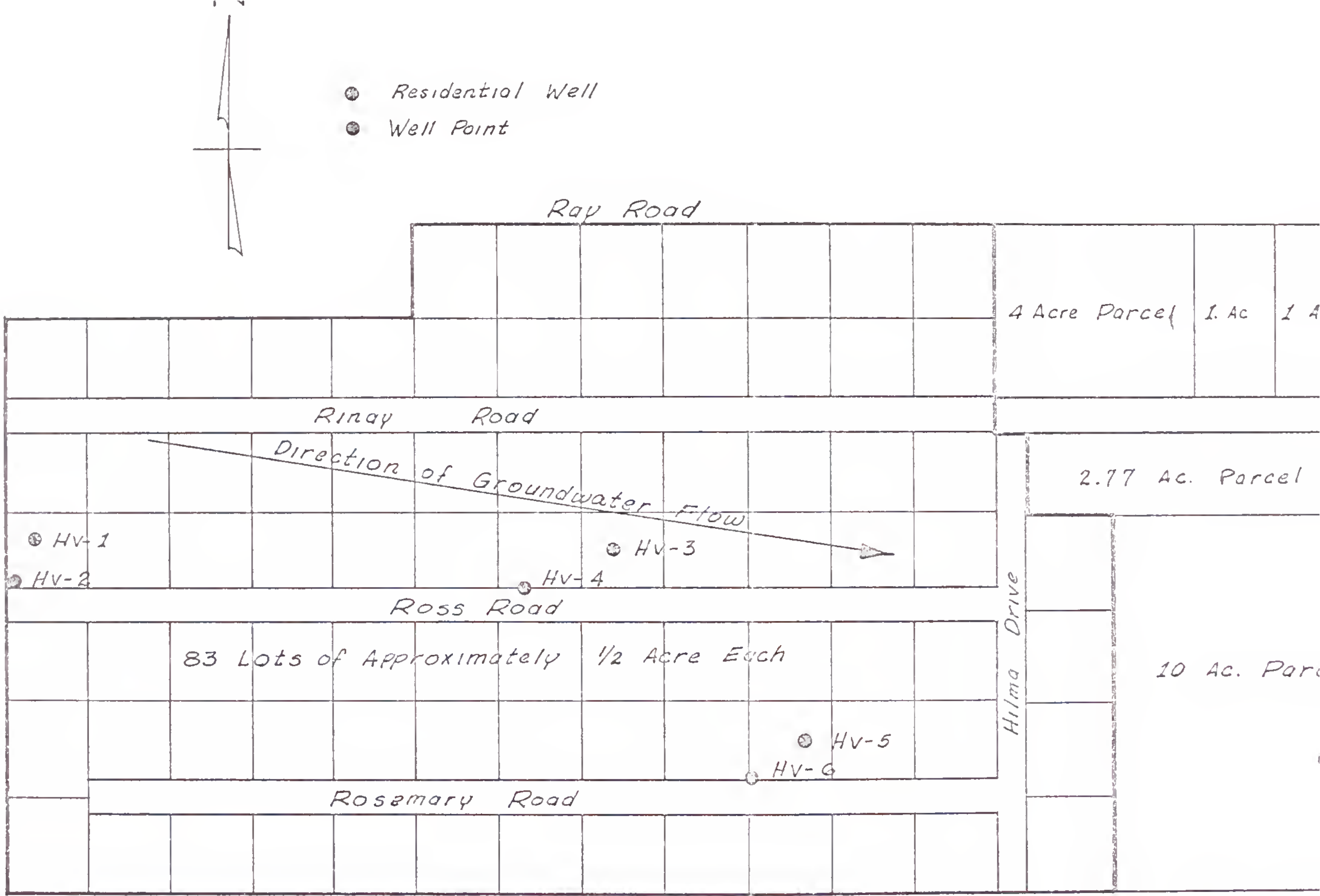
LAMBKIN'S ADDITION
LINCOLN, MONTANA

MAP NO. FOUR

FIVE ACRE TRACTS

LIVINGSTON, MONTANA




Residential Well
Well Point
Ray Road
4 Acre Parcel
1. Ac
1 A
Rinay Road
Direction of Groundwater Flow
2.77 Ac. Parcel
Hv-1
Hv-2
Hv-3
Hv-4
Ross Road
83 Lots of Approximately 1/2 Acre Each
Hilma Drive
10 Ac. Par
Hv-5
Hv-6
Rosamary Road

[illegible]